TENSION PILE STUDY

VOLUME V

IN SITU MODEL PILE EXPERIMENTS

IN SOFT CLAY

Report Number 82-200-6

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Report

to

Conoco Norway, Inc.

through

A.S. Veritec

Oslo, Norway

by

The Earth Technology Corporation

Houston, Texas

February 1986

The Earth Technology Corporation

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February 7, 1986

Conoco, Inc.
600 N. Dairy Ashford Rd
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Attention: Mr. Jack Chan

Re: CNRD 13-3 Tension Pile Study; Volume V - In Situ Model Pile Experiments in Soft Clay Report

Gentlemen:

We are pleased to submit herewith 20 copies of our Final report entitled "Tension Pile Study; Volume V - In Situ Model Pile Experiments in Soft Clay". Please forward the necessary copies to Conoco's staff, A.S. Veritec and your project participants, as you see appropriate.

We have enjoyed working on this project and look forward to be of further service to Conoco.

Very truly yours,

Jean M. E. Audibert, P.E.
Associate Principal
General Manager-Gulf States Division

Dewaine Bogard
Dewaine Bogard
Senior Engineer

JMEA/plm

cc: Rune Dahlberg, A.S. Veritec
Hudson Matlock

PREFACE

This report contains the results of eighteen experiments performed using 3-inch (7.62 cm) and 1.72-inch (4.37 cm)-diameter pile segment models at a decommissioned production platform in the West Delta Area, Gulf of Mexico. The work is part of a larger research project sponsored by Conoco, Inc. through Conoco Norway Inc. for the purpose of developing a better understanding of axial pile-soil interaction associated with foundation piles for tension leg platforms. The work was performed under two related projects, identified as CNRD 13-2 and CNRD 13-3. Also participating in support of the project were Chevron Oil Field Research Company, the American Bureau of Shipping, and the Minerals Management Service of the United States Department of the Interior.

The work was performed by the Earth Technology Corporation, acting as designated subcontractor to A.S Veritec (formerly Det Norske Veritas). The Earth Technology project team included the following staff members:

* Hudson Matlock, Vice President for Research and Development, provided overall technical direction for the project.

* Dewaine Bogard was responsible for project planning and administration, conceptual instrument design, the supervision of data collection and reduction, the interpretation of the results of the experiments, and the production of this report.

* Thomas K. Hamilton served as project manager for the CNRD 13-2 project, and assisted in project planning and administration during the early phases of the CNRD 13-3 project.

* Daniel K. Steussy was responsible for the mechanical design of the 3-inch (7.62 cm) diameter pile segment models.

* Ronald Boggess and Neil Dwyer designed and assembled the 1.72-inch (4.37 cm) diameter pile segment model and the data acquisition system.

* Thomas K. Hamilton, G. Leon Holloway, and Fleet Brown planned and supervised the installation of the pile segment models and the associated field activities.

* Chairat Suddhiprakarn performed most of the data reduction and the preparation of figures for this report.

* Jean Audibert, Manager of the Houston office, provided assistance in administrative matters and in the production and review of this report.

* Ignatius (Po) Lam and Lino Cheang of the Long Beach office also contributed to the success of the project.

The data acquisition software was developed by Mr. Bryan Fisher of Small Systems Solutions, Inc.

Portions of the work were handled directly by Conoco through parallel contracts. Offshore drilling operations and the deployment of the tools were performed by McClelland Engineers, Inc. Comet Construction Company of Venice, Louisiana, provided field and operational support.

Various representatives from Norway observed and assisted in the field tests. These included Tore Kvalstad, Kjell Hauge, and Rune Dahlberg of Veritec; Lars Grande of the Norwegian Institute of Technology, Trondheim; and Kjell Karlsrud of the Norwegian Geotechnical Institute.

Jack H. C. Chan, assisted by Jeff Mueller, served as Project Manager for Conoco, under the general direction of N. D. Birrell, Manager of Marine Engineering Division of PES, Conoco, Inc. R. L. Gratz, H. W. Wahl, and T. C. Ma served as project Administrators under the general direction of R. L. Mc Glasson and R. M. Vennet, General Managers of CNRD. George Santos and Tom Gautreaux of the

New Orleans Division of Conoco were responsible for platform modifications and preparation, and field support.

The first draft of this report was reviewed by Jean Audibert and Hudson Matlock. The final review of this report was performed by Jean Audibert.

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Research Projects 13-2 and 13-3. Additional reports will be issued which document the remainder of the work performed under the CNRD 13-3 research project.

The primary goal of the portion of the work reported herein was the collection of data from in situ experiments using small-diameter pile models for comparison with the results of the large-diameter pile load tests. The use of small-diameter tools for in situ testing allowed a much wider variation in the parameters thought to influence the development of shear transfer capacity along driven piles than would be possible with a large-diameter pile, including consolidation time and loading history.

The in situ tools were deployed at four different depths in six borings; comparisons of the behavior of the small-diameter models with that observed for the large-diameter pile at similar depths will aid in the understanding of axial pile-soil interaction.

In order to measure the soil parameters which were necessary to interpret the results of the experiments, the in situ tools were instrumented to obtain simultaneous measurements of the shear transfer, the local relative pile-soil displacement, the total radial soil pressure, and the pore water pressure. The measurements thus allowed the process of consolidation to be observed, and an evaluation of the relationships between the radial effective pressure and the shear transfer to be made.

The in situ tools which were developed were demonstrated to work very well. In only one experiment (of a total of eighteen which were performed) did a failure of the instruments prevent the successful completion of an experiment. For the major part of the work, the instruments exhibited excellent stability and repeatability; only during the long-term (3 month) period of consolidation did instrument drift result in uncertainty in the measurements, and then only in the values of total radial pressure.

Excellent agreement was obtained for the consolidation response which was observed during the experiments which were performed at each of the four depths; although there was some variation in the magnitudes of initial excess soil pressure which resulted from the installation of the tools, reasonable agreement among the final values of pore pressure and radial effective pressure were demonstrated.

Very good agreement was obtained among the values of peak shear transfer recorded during the experiments at each depth, especially among those values which were measured at the end of periods of undisturbed consolidation. In these tests, the value of peak shear transfer was shown to increase with the degree of consolidation, and thereby, the magnitude of the radial effective pressure.

Because of the agreement which was observed, the development of either a total stress approach (based on the degree of consolidation) or an effective stress approach (based on a slope-intercept relationship between shear transfer and the radial effective pressure) for the prediction of the ultimate static axial capacity appears reasonable.

During the cyclic load tests, the limiting values of shear transfer were observed to decrease with reversals of plastic slip, and to rapidly stabilize at values which were significantly smaller than the static ultimate shear transfer. Clear relationships could not be established between the values of cyclic minimum shear transfer and the degree of consolidation or the values of radial effective pressure which were based on soil pressures measured prior to or during the load tests.

Because of the rapidity and the magnitudes of the variations in the pore pressure (and thereby the radial effective pressure) during the cyclic tests, relationships among the instantaneous values of the radial effective pressure and the shear transfer were not felt to be meaningful. It was shown, however, that, under monotonic, undisturbed consolidation, the magnitude of the cyclic minimum shear transfer was essentially independent of both the degree of consolidation and the radial effective pressure, as measured prior to the load tests.

A consideration of the direction of the changes in pore pressure during cyclic loading, and of the effects of changes in void ratio and moisture content on the shear strength of clays, led to the conclusion that a portion of the reduction in shear transfer capacity during cyclic loading was likely to be associated with increases in the moisture content of the clay on and near a sharply-defined failure surface (although such a mechanism is not likely to be the sole factor involved in the cyclic degradation process).

During load tests which were performed in a number of the experiments at the end of varying periods of time after a series of initial cyclic load tests (and the associated cyclic degradation in resistance), values of peak shear transfer (and cyclic minimum shear transfer) were recorded which exceeded those measured in experiments at the same depths after comparable periods of undisturbed consolidation.

The results of such load tests, denoted as retests, led to the development of the concept of the superposition of two independent radial gradients in pore pressure, one from the initial installation and one from the shear-induced excess pore pressures from cyclic loading. Upon the dissipation of the combined excess pore pressures from both sources, increases in shear strength resulted which were larger than the increases which accompanied the dissipation of installation-induced excess pore pressures alone.

The physical examination of the clay which adhered to the tools after removal from the borings clearly indicated that failure occurred on a soil-soil surface, and that, during load tests which were performed after a period of reconsolidation following cyclic degradation, the second failure surface was at a greater radial distance from the pile wall than was the first. Thus, a part of the increase in shear transfer capacity during the retests was due to an outward shift in the failure surface and a resultant increase in the total shear force required to fail the soil, yielding an increase in the computed nominal shear stress on the surface of the probe.

In the absence of any quantifiable relationships among the degree of consolidation or the radial effective pressure and the values of the cyclic minimum shear transfer recorded during the load tests performed after periods of undisturbed consolidation, or the values of peak and cyclic minimum resistance recorded during the retests after periods of reconsolidation and recovery in resistance, an examination of the pressure fluctuations during cyclic loading led to a qualitative interpretation of the processes of cyclic degradation and the recovery in shear transfer capacity. Although the concepts were necessarily based on circumstantial evidence, the mechanisms of cyclic degradation and reconsolidation are consistent with established concepts of the shear strength of clay soils.

The results of the experiments thus indicate that, while the static ultimate axial pile capacity may be calculated using either a total stress or an effective stress approach, an estimation of the effects of cyclic loading on pile capacity must presently be empirical, with little concrete evidence upon which to base a quantitative approach.

The results of the experiments reported herein, combined with the results of the large-diameter pile tests, have provided a greatly-expanded base of high-quality data for use in the development and evaluation of existing and future axial pile capacity design methodologies.

1. INTRODUCTION

1.1 Introduction

This report presents the results of experiments performed at a decommissioned CAGC platform in Block 58 of the West Delta area, Gulf of Mexico. The experiments involve 3-inch (7.62 cm) and 1.72-inch (4.37 cm) diameter tools which simulate segments of an actual foundation pile and are instrumented to measure side friction, axial displacement, and lateral pressures. The probes are driven into undisturbed soil beyond the end of a borehole.

Twelve experiments were originally scheduled in the CNRD Research Projects 13-2 and 13-3; six experiments were added when the time became available in the schedule. Ten of the experiments were performed during the period from 30 November to 20 December 1982, under CNRD Research Project 13-2, with the remaining eight experiments being performed under CNRD Research Project 13-3 during the period from 12 December 1983 to 29 March 1984.

The experiments were a part of a comprehensive program of laboratory and field research on the axial behavior of piles driven into normally-consolidated clays and subjected to tension loading. Such piles would be used to anchor a Tension Leg Platform in deep water sites off the Mississippi Delta in the Gulf of Mexico. Although the experiments were performed for the specific purpose of examining the behavior of axially loaded friction piles under tension loading, the results are equally applicable to friction piles subjected to compression loading.

The overall project organization is shown in Plate 1.1. The primary sponsorship was by Conoco, Inc., through Conoco Norway Research and Development (CNRD) of Conoco Norway, Inc., with participation by Chevron Oil Company, the American Bureau of Shipping, and the Minerals Management Service of the United States Department of the Interior.

1.2 Contents of this Report

This report contains the results of all the pile segment experiments at the West Delta 58A platform site. Only the results of the experiments will be included; the correlation of these results with the soil properties at the site and with the results of a test on an instrumented 30-inch (76.2 cm) diameter pile will be covered in a subsequent report.

The pile segment models were instrumented to simultaneously measure the shear transfer, displacement, radial total pressure, and pore water pressure during the insertion, set-up consolidation, and axial loading of the probes. The probes were deployed beyond the bottom of boreholes at various depths. A through-hole hydraulic cylinder attached to the top of a conventional N-rod string was used to apply axial loads to the models.

The program of experiments included load tests at various stages in the consolidation history, the application of different load histories at equal consolidation times, and the retesting of the probes at various times after an initial cyclic load test to investigate the process of self-healing of the soil after cyclic degradation.

The data from these experiments will be combined with results from a parallel program of laboratory experiments on 1-inch (2.54 cm) diameter models (performed by A.S Veritec) and from tests (performed by The Earth Technology Corporation) on a 30-inch (76.2 cm) diameter instrumented pile at the West Delta platform to study pile-soil interaction, including evaluation of effective stress analyses of axial pile capacity.

1.3 Organization of this Report

A brief description of the soil conditions at the site are given in Chapter 2 of this report. For a more detailed description of the soils, the reader is referred to Ref 1.

Chapter 3 of the report contains a technical description of the in situ tools, the loading system, and the data acquisition system.

Chapter 4 contains a description of the methods used to perform the experiments, and an overview of the experiments performed during the course of the work.

The results of the experiments are given in Chapter 5. Due to the large volume of data accumulated during the eighteen experiments, the data are presented in a distilled and condensed form. The records include all the data taken during the course of the work; however, only those results which are felt to be of most significance in establishing methods of practical design are discussed.

The results of the experiments, and the conclusions drawn from the small-diameter pile segment model experiments, are summarized in Chapter 6.

2. SITE DESCRIPTION

2.1 General

A map showing the location of the offshore test site is given in Plate 2.1. The water depth at the site is approximately 49 ft (14.9 m). A generally homogeneous clay stratum, having no sand or silt layers in the zone of interest, extends from the seafloor to a depth of 253 ft (77.1 m).

The stratigraphy of interest (above the 234 ft (71.3 m) depth) can be described as an olive gray clay, being very soft at the seafloor and increasing in strength to a stiff consistency at a depth of 253 ft (77.1 m). The stratigraphy is shown in more detail in the boring logs contained in Plates 2.2, 2.3, and 2.4, which were taken from Ref 1.

The interpreted shear strength profile, given in Plate 2.4, is bilinear in nature, with a shear strength of 0.1 ksf (4.8 kPa) at the mudline, which increases to 0.75 ksf (35.9 kPa) at a depth of 160 ft (48.8 m), followed by a linear increase to 1.80 ksf (86.2 kPa) at a depth of 250 ft (76.2 m).

The interpreted unit weight profile shown in Plate 2.5 was derived from the results of both field and laboratory tests (Ref 1).

The results of Atterberg limit tests on samples taken from the site are given in Plate 2.6 and confirm that the clays have a high activity.

Based on the results of the laboratory and field tests, the clay deposit at the site has been divided into three strata (Ref 1), with the three strata being approximately bounded by the depths of 0 to 80 ft (0 to 24.4 m), 80 to 160 ft (24.4 to 48.8 m), and 160 to 253 ft (48.8 to 77.1 m) below the mudline.

3. TEST EQUIPMENT AND INSTRUMENTATION

3.1 Introduction

A description of the in situ probes and data acquisition equipment will be contained in this section of the report.

3.2 The 3-inch (7.62 cm)-diameter Pile Segment

The 3-inch-diameter (7.62 cm) pile segment model is shown schematically in Plate 3.1. The shear transfer is measured by two axial load cells placed 30.56 inches (77.62 cm) center-to-center, arranged so as to have a surface area of 2 sq ft (1858 cm^2) between the load-measurement points. The strain gages are arranged so that the strain gage bridges are sensitive only to axial load, with the axial strains due to the effects of external pressure and bending being self-cancelling.

A total of eight strain gages are placed on each load cell, with alternate strain gages being connected in series, forming two pairs of four strain gages each on each load cell. The pairs of four strain gages from each load cell are then connected with those from the second load cell to form a fully-active Wheatstone bridge, with each load cell providing two opposite arms. In this manner, the output from the strain gage bridge will be proportional to the total change in load between the two load cell cross-sections at which the strain gages are placed, with common axial loads being self-cancelling, provided that the two load cells have identical mechanical (AE) and electrical resistance properties. In practice, the output from the pair of load cells in each probe differed by less than one percent. The calibration factor which was used to convert output voltage to shear transfer was calculated from the calibrations of each individual load cell, with any differences in the sensitivity of the two load cells in a probe being accounted for in the calculations.

The radial total pressure is measured by a small load cell which is arranged so that the active face passes through the wall of the transducer housing, with the exposed face being shaped to conform to the outer surface of the pile segment model. O-ring seals are placed around the active face to prevent pressure losses from occurring across the wall of the transducer housing.

The total pressure transducers were calibrated using dead-weight loads measured by a proving ring. The loads were converted to equivalent pressures based on the dimensions of the active face. In addition, the calibration factors were checked by placing the probe in a pressure chamber filled with water and recording the output as a function of the applied hydrostatic pressure.

The pore water pressure was measured with commercially-available Statham PA856-500 pressure transducers. The transducers were mounted in a machined housing, with a disc-shaped porous stone serving as a filter to absorb the radial effective pressure and to transmit only the pore fluid pressure to the cavity behind the porous stone. Prior to deploying the probes, the porous stones were saturated by boiling them in de-aired water. The cavity behind the porous stone was then filled with de-aired water, the porous stone inserted, and the probe quickly lowered below the water surface to prevent water evaporation and desaturation of the filter.

The pore pressure transducers were calibrated by placing the assembled probes together with their cables in a 30-inch (76.2 cm) diameter, 10-ft (3.0 m) long pressure vessel, filling the vessel with water, and applying a series of known hydrostatic pressures to the water. The applied pressures were read with a precise Bourdon gage. At the same time, the output from the pressure transducers were read, so that the calibration factors of both types of pressure transducers could be verified against identical values of pressure.

During load tests, the relative displacement of the probe was measured using a DC-LVDT, Schaevitz Engineering model number GCD-121-500. The body of the LVDT was securely mounted in a housing at the bottom of the lower load cell, with the core of the transducer being attached to a connecting assembly which held the cutting shoe. The connecting assembly was attached to the probe through a slip joint, which allowed the probe body to move freely with respect to the lower unit. The output of the LVDT was thus proportional to the relative movement between the probe body and the cutting shoe, and thereby, the adjacent soil. The slip joint was designed to allow one inch (25.4 mm) of relative movement between the two sections, with mechanical stops provided so that the probe and cutting shoe could be deployed and retrieved as a single assembly.

3.3 The 1.72-inch (4.37 cm)-diameter Pile Segment Model (The X-probe)

The 1.72-inch (4.37 cm) diameter pile segment model is shown schematically in Plate 3.2.

The upper housing on the probe contains the electronics package which includes the electronic circuits necessary to power the transducers and to amplify the output of the strain gage bridges. Prior to deployment of the probe, the stability of the circuits was verified by monitoring the output of the transducers for several days, with no noticeable drift being observed in the output.

The shear transfer is measured by a load cell which supports a cylindrical outer sleeve. The outer sleeve is attached to the probe only at its connection to the load cell, thus giving a direct measure of the average shear transfer on the 31 sq inches (200 sq cm) surface area of the sleeve.

The total radial pressure is measured by a load cell which supports two active faces, one at each end of the load cell. The load cell is free-floating in a cylindrical cavity. The active faces pass through the wall of the transducer housing, and are machined to conform to the cylindrical surface of the pile segment model.

The pore pressure is measured by a Statham PA856-500 pressure transducer. The transducer is mounted in a housing which has a cylindrical inner cavity connected to the outer surface of the probe through three holes. At the probe surface, a milled hole 3/16-inch (4.8 mm) deep and 5/8-inch (15.9 mm) in diameter intersects each of the three paths to the probe interior. Porous stones are pressed into the three milled holes, and their outer surfaces are dressed to conform to the surface of the probe.

The axial displacement of the probe relative to the soil is measured by a DC-LVDT, Schaevitz Engineering model number GCD-121-500. The body of the LVDT is attached to the probe body, and the core of the LVDT is attached to a soil anchor which is connected to the probe through a slip joint. The length of the soil anchor can be varied so as to provide the necessary anchoring resistance in soil strata of differing shear strengths.

The transducers on the probe are calibrated in a specially designed cylindrical pressure chamber. A disc-shaped annular sleeve, with O-ring seals, is mounted on the friction sleeve of the probe. When the probe and sleeve are inserted into the pressure chamber, the annular sleeve forms a hydraulic piston. The body of the probe is restrained from motion at each end, and end caps are attached to both ends of the pressure chamber. The two cavities thus produced are filled with water, and hydraulic pressure is sequentially applied to each chamber.

The applied pressure is converted to a load, and then to an equivalent value of shear transfer, in order to obtain a factor by which the output voltage of the load cell can be related to shear transfer. At the same time, the total pressure and pore pressure transducers are read and calibrated, with the values of pressure being read on a precise Bourdon gage.

3.4 The Data Acquisition System

The data acquisition system is shown schematically in Plate 3.3. The system is designed to utilize a Hewlett-Packard 3497A Scanning Digital Voltmeter as an analog-to-digital converter. The HP3497A, with 6-1/2 digit accuracy, was selected in order to read strain-gage level signals directly, without intermediate signal conditioning equipment.

The computer programs which control the data acquisition equipment were written so that each set of four data channels devoted to any probe could be independently sampled, with up to four probes being sampled in parallel. Each probe could be sampled at a rate which was convenient for the particular function being performed. During consolidation, the scan rates varied from 10 seconds to 2 hours between samples. During the load tests, the sampling rate was increased to a maximum of one sample per second, with a typical sample rate of one scan each 3 to 5 seconds.

The four channels of data were sequentially sampled, digitized, and passed to a Digital Equipment Corporation MINC 11/23 computer for visual display and for storage on 8-inch (20.3 cm) floppy discs. The digital records stored on the floppy discs were transferred to a 9-track magnetic tape as each floppy disc was filled, or at a convenient time during the testing program, e.g., when the necessary time could be spared between samples. The magtape is used both for redundancy and for transferring the digital records to non-DEC computers for post-processing.

After each scan, or sample of data from a probe, the raw voltages were converted into engineering units, and the results printed. The printout was used as a second level of redundancy, and to serve as a later guide to the tests, since provision had also been made for the immediate printing of short (one line) comments from the operator during the course of the experiments.

Parallel analog records of the tests were made on x-y-y recorders. The analog records were used for control and immediate visualization of the progress of a test. The analog records normally included shear transfer versus displacement on one recorder, with the pressure variations versus time being recorded on the second.

3.5 The Portable Loading and Control System

The portable loading system is shown schematically in Plate 3.4. The load is applied to a short section of N-rod which has a circular flange welded at one end. The displacement of the hydraulic ram is transferred to the probes by N-rods, through which the instrument cables are passed to the surface.

The hydraulic ram is controlled by one-directional flow control valves, which are mounted on the upper steel plate. The direction of travel of the ram is controlled by an electrical 4-way solenoid valve which is also mounted on the upper steel plate. The solenoid valve can be actuated manually, by micro-switches mounted on the analog recorder, or by a signal actuated by the digital output from the voltmeter, upon request by the MINC 11/23 computer. During the course of the experiments, both methods of control were used, in order to best perform each specific type of load test.

In addition to the solenoid valve which controlled the direction of travel, a second solenoid valve, actuated manually, permitted flow through two additional flow-control valves which made up a parallel hydraulic circuit. The parallel hydraulic circuit was used for fast-rate loading and for rapid adjustment of ram height when connecting the loading system to the N-rod string.

The nominal rate of travel of the ram was approximately 0.001 inch (0.025 mm) per second during the quasi-static load tests. The rate of displacement of the probes varied considerably during some of the load tests, since the vertical motion of the structure associated with tilting due to wave loading either added to, or subtracted from, the ram motion. During some of the tests, the motion of the structure resulted in full reversal of probe movement, even during plastic slip, as shown in Plate 3.5. During one rest period, vertical motions of a probe of only 50×10^{-6} inch (0.0013 mm) were clearly detected and displayed on the x-y plotter.

The effects of the motion of the structure on the digital data was the creation of an apparent band of noise or uncertainty. The digital data points, obtained at equal intervals of time, load, or displacement, fell on random peaks and valleys of the analog records, such as that shown in Plate 3.5. The digital records reflected the variation in real physical quantities, as the structural motion and the resultant vertical movements imparted to the probes caused variations in the shear transfer, displacement, total and pore pressure in the soil adjacent to the moving probe. With the exception of two probes which developed leakage in the connector, the output from all the instruments was extremely stable, with noise levels considerably smaller than the changes in the measured variables caused by the motion of the structure, as can be seen in Plate 3.5.

3.6 Methods of Installation

The sequence of events during the installation of a 3-inch-diameter (7.62 cm) probe are shown in Plate 3.6. The events are shown in the figure left to right in order of occurrence.

Prior to installing the tools in a boring, the boring was advanced to a depth of approximately 4 feet (1.2 m) above the desired test depth, as shown on the left. The drill bit is then retracted 3.5 feet (1.1 m) above the bottom of the borehole. The probe is then lowered into the borehole until the tip of the cutting shoe encounters the soil. The N-rod string is then retracted a few inches, and the strain gage bridges monitored for several minutes, in order to ascertain that the output voltages from the strain gage bridges are stable, and that no leakage of moisture has occurred.

The probe is then lowered until the cutting shoe penetrates the soil a sufficient distance to support the probe and the drill-rod string. A 300-lb (1.33 kN) casing hammer is then used to apply impact forces to the top of the N-rod string, advancing the probe into the soil below the borehole. As the cutting shoe advances into the soil, the thin-walled tube acts as a cookie cutter, simulating an open-ended pipe pile, and creating excess lateral earth pressures which are consistent with those developed along prototype-sized piles having the same diameter to wall thickness ratio.

At the time the cutting shoe has advanced into the soil to a depth of 7 feet (2.1 m), as shown in the middle drawing, the soil plug encounters the solid (except for a small weep-hole) metal adaptor. During the remainder of the driving operation, the cutting shoe is advanced in a fully-plugged, or closed-end condition.

At the end of driving, as shown in the drawing on the right, the instrumented section resides near the center of the soil layer through which the cutting shoe advanced prior to plugging. Thus, the zone of soil in which the measurements of shear transfer and lateral pressure are made is one with low excess lateral earth pressures. The probes were designed so that the top-most load cell was located a distance of ten probe diameters below the bottom of the borehole, and the bottom load cell a distance of eight diameters above the depth at which the cutting shoe plugged, in order to isolate the sensitive section from the complex pressure variations which occur at both locations.

Similar procedures were followed with the 1.72-inch (4.37 cm)-diameter tool, except that the probe was advanced to the test depth by pushing, rather than driving. Since the weight of the N-rod string was more than sufficient to push the probe into the soil, the N-rod string was supported at deck level at all times.

4. PROGRAM OF EXPERIMENTS

4.1 Introduction

The purpose of the experimental program was to collect data from small-diameter pile segment models at various depths in the three strata at the West Delta 58A platform site for comparison and correlation with the results of the 30-inch (76.2 cm) diameter pile load test and with the laboratory experiments performed on 1-inch (2.54 cm) diameter models by A.S Veritec (formerly Det Norske Veritas) in remolded samples of the same clay.

The periods allowed for consolidation prior to load testing the probes was varied through a wide range, in order to collect as much data as possible to relate the increases in shear resistance to the degree of consolidation and to the radial effective pressure. In addition, rest periods of varying lengths were allowed after the load tests at intermediate degrees of consolidation, in order to investigate the process of reconsolidation and recovery of shear transfer capacity by the soil.

4.2 Chronological Sequence of Data Within Each Experiment

The experiments with the small-diameter probes can be divided into a number of separate categories, taken in the chronological sequence in which the events occurred. Each experiment included the following:

1. Installation, either by driving (3-inch (7.62 cm) diameter probes) or by pushing (1.72-inch (4.37 cm) diameter probe)

2. A load test to failure in tension and compression within minutes after installation, ending with approximately 1/2 inch (13 mm) of travel to center the LVDT

3. A period of uninterrupted consolidation, to a desired degree of reduction of the initial excess pore pressure

4. A quasi-static and a cyclic load test, with the desired load history, including both load and displacement-controlled testing at levels of load or displacement less than that required for plastic failure in the soil

5. A large-displacement cyclic test, with fully-reversed plastic slip, during which the maximum amount of cyclic degradation in resistance is achieved

6. An optional (and variable) period of reconsolidation and recovery in resistance

7. A load test to failure in tension, optionally followed by a cyclic test to failure at large displacements

The program of load tests was planned to allow for the observation of the development of shear transfer capacity and the increase in the radial effective pressure with time after driving. The load tests were performed to measure the maximum shear transfer under monotonic loading to failure and to observe the maximum degradation in resistance under reversal of large displacements, with parallel small-displacement or repeated-tension loadings at pre-failure load levels in order to observe the initiation of cyclic degradation in resistance. The reconsolidation and retests of the probes were done to examine the effects of time and the dissipation of shear-generated excess pore pressures on the recovery in axial resistance after cyclic degradation.

For the long-term tests, the consolidation during the experiments occurred with the soil around the probes in three conditions; fully degraded early in the consolidation process, fully degraded late in the consolidation process, and under a long period of undisturbed consolidation. Unfortunately, the probe which was tested early in the consolidation process developed leakage into the connector, and was not load-tested after the long period of consolidation.

4.3 Log of Pile-Segment Model Experiments

The experiments were performed at depths of 58, 148, 178, and 208 ft (17.7, 45.1, 54.3, and 63.4 m) below the mudline. In the following description of the experiments which were performed, the term "experiment" refers to an independent installation of a probe at some depth; the terms "test" and "tests" refer to one or several active loadings of a probe performed after installation at that depth.

In addition to the load tests which were performed at various times after installation, as described below, a load test to failure in tension and compression was performed immediately (within minutes) after the installation of each probe in each experiment.

Three independent experiments were performed at the 58-ft (17.7 m) depth:

1. A large-displacement, two-way cyclic test after 72 hours 3 minutes of consolidation

2. A large-displacement, two-way cyclic test after 6 hours 58 minutes of consolidation, with one cycle of loading to failure in tension and compression after 19 hours 25 minutes of consolidation

3. A progressively-increased, symmetric, displacement-controlled test after 8 hours 26 minutes of consolidation, with a large-displacement, two-way cyclic test after 19 hours 10 minutes of consolidation

The three experiments thus yielded comparisons of the effects of consolidation time on the maximum and cyclic minimum resistances (Experiments 1 and 2), a comparison of the value of shear transfer at the initiation of cyclic degradation to the cyclic minimum resistance under immediate large-displacement cyclic loading (Experiments 2 and 3), and two observations of the recovery of shear resistance with time after cyclic degradation (Experiments 2 and 3).

The chronological log of the load tests performed during the experiments at the 58-ft (17.7 m) depth is given in the following Table. The periods of consolidation, and of reconsolidation and recovery, may be determined by examining the time intervals between the load tests.

TABLE 4.1 CHRONOLOGICAL SEQUENCE OF EXPERIMENTS AT THE 58-FT DEPTH

EXPERIMENT	DATE	TIME	EVENT
1	30 Nov 1982	19:41	End of driving
		19:50	Begin Immediate load test (Duration 8 minutes)
	03 Dec 1982	19:44	Static load test after consolidation
		19:48	Begin large-displacement cyclic test
		20:33	End of large-displacement cyclic test
		20:49	Probe removed from boring
2	01 Dec 1982	11:38	End of driving
		11:49	Begin Immediate load test (Duration 11 minutes)
		19:35	Begin large-displacement cyclic test
		20:19	End of large-displacement cyclic test
	02 Dec 1982	07:03	Retest after reconsolidation (Duration 7 minutes)
		07:28	Probe removed from boring
3	05 Dec 1982	11:26	End of driving
		11:32	Begin Immediate load test (Duration 15 minutes)
		19:52	Begin Displacement-Controlled cyclic tests
		23:29	End of cyclic tests
	06 Dec 1982	06:36	Begin retest (Duration 24 minutes)
		07:13	Probe removed from boring

Five independent experiments were performed at the 148-ft (45.1 m) depth:

1. A large-displacement, two-way cyclic test after 68 hours 1 minute of consolidation

2. A large-displacement, two-way cyclic test after 8 hours 34 minutes of consolidation, with one cycle of loading to failure in tension and compression after 16 hours 53 minutes of consolidation

3. Large-displacement, two-way cyclic tests after 8 hours 21 minutes, 20 hours 28 minutes, and 43 hours 58 minutes of consolidation

4. Large-displacement, two-way cyclic tests after 4 hours 15 minutes, 44 hours 19 minutes, and 96 hours 22 minutes of consolidation

5. An experiment with the 1.72-inch (4.37 cm) diameter probe, with large-displacement two-way cyclic tests after 1 hour 15 minutes and 25 hours of consolidation

The experiments thus yielded a comparison of the effects of consolidation time on the development of shear transfer (Experiments 1, 2, 3, and 4), an observation of the effects of diameter and plugging on the consolidation and shear transfer (Experiment 5), and the observation of the effects of time and consolidation on the recovery of resistance after cyclic degradation (Experiments 2, 3, 4, and 5).

The chronological sequence of the load tests performed during the experiments at the 148-ft (45.1 m) are given in the table below. The periods of consolidation, and of reconsolidation and recovery in resistance, may be determined from the time intervals between each load test in each experiment.

TABLE 4.2 CHRONOLOGICAL SEQUENCE OF EXPERIMENTS AT THE 148-FT DEPTH

EXPERIMENT	DATE	TIME	EVENT
1	04 Dec 1982	17:18	End of driving
		17:29	Begin Immediate load test (Duration 11 minutes)
	07 Dec 1982	13:19	Static load test (Duration 3 minutes)
		13:23	Begin large-displacement cyclic test
		14:01	End of large-displacement cyclic test
		14:10	Variable-rate test (Duration 6 minutes)
		14:32	Probe removed from boring
2	02 Dec 1982	14:10	End of driving
		14:18	Begin Immediate load test (Duration 9 minutes)
		22:44	Begin large-displacement cyclic test
	03 Dec 1982	00:20	End of large-displacement cyclic test
		07:03	Begin retest (Duration 12 minutes)
		07:30	Probe removed from boring
3	06 Dec 1982	11:46	End of driving
		11:52	Begin Immediate load test (Duration 9 minutes)
		20:07	Begin large-displacement cyclic test

		20:54	End of large-displacement cyclic test
	07 Dec 1982	08:14	Begin first large-displacement cyclic retest
		08:52	End of first large-displacement cyclic retest
	08 Dec 1892	07:44	Begin second large-displacement cyclic retest
		08:11	End of second large-displacement cyclic retest
		08:12	Begin variable-rate test (Duration 13 minutes)
		08:38	Probe removed from boring
4	12 Dec 1983	14:50	End of driving
		15:05	Begin Immediate load tests (Duration 19 minutes)
		19:05	Begin large-displacement cyclic test
		20:18	End of large-displacement cyclic test
	14 Dec 1983	11:09	Begin first large-displacement cyclic retest
		12:22	End of first large-displacement cyclic retest
	16 Dec 1983	15:12	Begin second large-displacement cyclic retest
		16:23	End of second large-displacement cyclic retest
		16:42	Probe removed from boring

5	13 Dec 1983	12:34	X-probe pushed to grade
		12:51	Begin Immediate load test (Duration 15 minutes)
		13:49	Begin large-displacement cyclic test
		14:41	End of large-displacement cyclic test
	14 Dec 1983	13:34	Begin large-displacement cyclic retest
		14:15	End of large-displacement cyclic retest
		14:52	Probe removed from boring

Four independent experiments were performed at the 178-ft (54.3 m) depth:

1. A load-controlled, one-way repeated load test followed by a large-displacement, two-way cyclic test after 8 hours 24 minutes of consolidation, followed by a large-displacement, two-way cyclic test after 19 hours 33 minutes of consolidation

2. A large-displacement, two-way cyclic test after 8 hours 16 minutes of consolidation

3. A large-displacement, two-way cyclic test after 71 hours 30 minutes of consolidation

4. An experiment with the 1.72-inch (4.37 cm) diameter probe, with large-displacement, two-way cyclic tests after 24 hours 2 minutes and 48 hours 47 minutes of consolidation

The experiments thus yielded a comparison of the effects of consolidation time on the maximum and cyclic minimum shear transfer (Experiments 2 and 3), a comparison of the value of shear transfer at the initiation of cyclic degradation to the cyclic minimum resistance with immediate large-displacement cyclic loading (Experiments 1 and 2), an observation of the effects of diameter and end-plugging on consolidation and shear transfer (Experiment 4), and two observations of the effects of time and reconsolidation on the recovery in shear resistance after cyclic degradation (Experiments 1 and 4).

The chronological history of the load tests performed during each experiment at the 178-ft (54.3 m) depth are given in the following table.

TABLE 4.3 CHRONOLOGICAL SEQUENCE OF EXPERIMENTS AT THE 178-FT DEPTH

EXPERIMENT	DATE	TIME	EVENT
1	08 Dec 1982	11:44	End of driving
		11:51	Begin Immediate load test (Duration 9 minutes)
		20:08	Begin load-controlled cyclic tests (with large-displacement cyclic test after initial tensile pullout)
		23:23	End of large-displacement cyclic test
	09 Dec 1982	07:18	Begin large-displacement cyclic retest
		07:59	End of large-displacement cyclic retest
		08:01	Variable-rate test (Duration 8 minutes)
		08:22	Probe removed from boring
2	17 Dec 1983	12:28	End of driving
		12:36	Begin Immediate load test (Duration 8 minutes)
		20:44	Begin large-displacement cyclic test
		21:53	End of large-displacement cyclic test
	18 Dec 1983	07:03	Probe removed from boring without load testing
3	13 Dec 1983	17:15	End of driving

		17:23	Begin Immediate load test (Duration 12 minutes)
	16 Dec 1983	16:45	Begin large-displacement cyclic test
		17:56	End of large-displacement cyclic test
		18:15	Probe removed from boring
4	14 Dec 1983	17:52	X-probe pushed to grade
		17:52	Begin Immediate load test (Duration 7 minutes)
	15 Dec 1983	17:54	Begin large-displacement cyclic test
		18:47	End of large-displacement cyclic test
	16 Dec 1983	18:39	Begin large-displacement cyclic retest
		19:16	End of large-displacement cyclic retest
		20:00	Probe removed from boring

Six independent experiments were performed at the 208-ft (63.4 m) depth:

1. A large-displacement, two-way cyclic test after 69 hours 18 minutes of consolidation

2. Large-displacement, two-way cyclic tests after 8 hours 30 minutes, 69 hours 49 minutes, and 143 hours 13 minutes of consolidation

3. A progressively-increased, symmetric, displacement-controlled cyclic test after 7 hours 57 minutes of consolidation, with one cycle of loading to failure in tension and compression after 18 hours 42 minutes of consolidation

4. A large-displacement, two-way cyclic test after 7 hours 57 minutes of consolidation, followed by 102 days of consolidation (the pressure transducer output voltages exhibited instability after 24 hours)

5. Large-displacement, two-way cyclic tests after 72 hours 32 minutes and 2479 hours 26 minutes of consolidation

6. A large-displacement, two-way cyclic test after 2467 hours 9 minutes of consolidation

The experiments thus yielded a comparison of the effects of undisturbed consolidation on the development of maximum and minimum shear transfer (Experiments 1, 2, 4, 5, and 6), a comparison of the value of shear transfer at the initiation of cyclic degradation to the cyclic minimum resistance (Experiments 2, 3, and 4), and observations of the recovery in shear resistance with time after cyclic degradation (Experiments 2, 3, and 5).

A chronological history of each experiment, including each load test performed during each experiment, is given in the following table.

TABLE 4.4 CHRONOLOGICAL SEQUENCE OF EXPERIMENTS AT THE 208-FT DEPTH

EXPERIMENT	DATE	TIME	EVENT
1	07 Dec 1982	19:22	End of driving
		19:29	Begin Immediate load test (Duration 10 minutes)
	10 Dec 1982	16:40	Static load test (Duration 2 minutes)
		16:44	Begin large-displacement cyclic test
		17:21	End of large-displacement cyclic test
		20:33	Probe removed from boring
2	03 Dec 1982	12:39	End of driving
		12:52	Begin Immediate load test (Duration 10 minutes)
		21:09	Static load test (Duration 4 minutes)
		21:13	Begin large-displacement cyclic test
		21:49	End of large-displacement cyclic test
	06 Dec 1982	10:28	Begin first retest (Duration 13 minutes)
		10:42	Begin first large-displacement cyclic retest
		11:10	End of first large-displacement cyclic retest
	09 Dec 1982	11:52	Begin second large-displacement cyclic retest

		12:40	End of second large-displacement cyclic retest
		12:42	Begin variable-rate load test (Duration 11 minutes)
	10 Dec 1982	14:27	Probe removed from boring without load testing
3	09 Dec 1982	13:24	End of driving
		13:30	Begin Immediate load test (Duration 10 minutes)
	09 Dec 1982	21:21	Begin small-displacement cyclic tests (ending with large-displacement test)
	10 Dec 1982	01:12	End of large-displacement cyclic test
		07:56	Single cycle of reversed large-displacement loading (Duration 11 minutes)
		08:21	Probe removed from boring
4	18 Dec 1983	13:55	End of driving
		14:04	Begin Immediate load test (Duration 18 minutes)
		21:52	Begin large-displacement cyclic test
		23:30	End of large-displacement cyclic test
	29 Mar 1984	18:36	Probe removed from boring without load testing
5	17 Dec 1983	00:26	End of driving

		00:38	Begin Immediate load test (Duration 13 minutes)
	20 Dec 1983	00:58	Begin large-displacement cyclic test
		02:39	End of large-displacement cyclic test
	29 Mar 1984	07:52	Begin large-displacement cyclic retest
		09:29	End of large-displacement cyclic retest
		09:33	Begin variable-rate test (Duration 6 minutes)
		10:11	Begin second variable-rate test (Duration 14 minutes)
		15:19	Probe removed from boring
6	17 Dec 1983	16:36	End of Driving
		16:45	Begin Immediate load test (Duration 19 minutes)
	29 Mar 1984	11:45	Begin large-displacement cyclic test
		12:42	End of large-displacement cyclic test
		13:23	Begin variable-rate test (Duration 16 minutes)
		13:49	Begin second variable-rate test (Duration 11 minutes)
		20:07	Probe removed from boring

During the installation of the 3-inch (7.62 cm) diameter probes in four of the experiments, including all three experiments at the 58-ft (17.7 m) depth and Experiment 3 at the 148-ft (45.1 m) depth, the pre-drilling of the borehole was stopped 3.5 ft (1.1 m) short of the planned depth (See Plate 3.6). For these experiments, the cutting shoe plugged when the tip of the shoe was approximately at the depth where the pressure transducers became located at the end of driving. Thus, the magnitudes of the excess soil pressures which were developed represented an intermediate condition, somewhere between a thin-wall and a fully-plugged end condition.

Although the intermediate values of excess pressure do not properly reflect the desired set of boundary conditions for consolidation, the later performance of an experiment with the X-probe, with fully-plugged end conditions, at the 148-ft (45.1 m) depth aided in the interpretation of the results of Experiment 3 at this depth, and also the experiments at the 58-ft (17.7 m) depth.

The two programs of research performed at the offshore site with the small-diameter in situ probes have provided much valuable information on axial soil-pile interaction. The extremely wide range in consolidation time, with the soil in both disturbed and undisturbed conditions resulting from cyclic load tests, will provide the basis for a much-improved understanding of the mechanics of the axial behavior of piles.

Coupled with the results of the research using the 30-inch (76.2 cm) diameter pile, and the parallel analytical developments, a greatly improved and expanded basis for axial pile design has been obtained.

5. RESULTS OF THE FIELD EXPERIMENTS

5.1 Introduction

The results of the experiments will be presented in this section of the report in increasing order of depth, without regard for the chronological sequence in which the experiments were performed. This format will be followed so that the results of the experiments can be compared and assimilated more easily.

For the most part, only the digital data will be presented.

For some of the load tests, primarily those in which the structural motion due to wave loading prevented the digital data acquisition system from obtaining an accurate representation of the shear transfer versus displacement relationships, the digital data was augmented or replaced with that obtained using the analog recorders.

During other load tests, the digital system did not obtain sufficient data points to accurately reflect the behavior of the soil. For these tests, the analog data was used to supplement the data points obtained by the digital data acquisition system, so that the curve shapes which are presented herein are an accurate reproduction of the response of the soil, as recorded by the analog (x-y) plotter.

In the presentation of the results of the load tests, the complete displacement history of the probes have not been shown. Of more concern for design are the effects of cyclic loading on the magnitudes of shear transfer and on changes in the deformational characteristics of the soil. Therefore, only the first and last cycles of loading have been included, with the results shown in the form of quasi-static shear transfer versus displacement relationships.

The complete history of loading is given in the form of a plot of the variations in the shear transfer, the pore pressure, and the radial effective pressure with time during the load test. This was done for two purposes: 1) to relate the changes in pore pressure to the process of cyclic degradation, and 2) to provide the necessary information to obtain the input parameters for axial pile analysis programs.

The discussion will be limited to those results which are felt to be of greatest practical significance, without undue emphasis being placed on those aspects of behavior which, although of academic interest, do not contribute to the technical or practical significance of the results.

It is intended to provide sufficient detail to the data presented in this chapter so that the results given herein may be used for correlation and comparison with present and future analytical procedures and design methods. Thus, the discussion of the results of the individual experiments is quite lengthy, and may not be of general interest.

A comprehensive summary of the results of the experiments is given in Chapter 6; the discussion includes a summary of the experiments presented in detail in Chapter 5, and thus provides a succinct account of the more important findings of the work.

5.2 The Experiments at the 58-ft (17.7 m) Depth

Since the primary source of soil resistance to axial pile loading (and the zone of primary interest in the 30-inch (76.2 cm)-diameter pile test) is the soil at large depths, the major part of the work was performed in the depths below 148 ft (45.1 m), with fifteen of the eighteen experiments being performed at depths of 148 (45.1 m), 178 (54.3 m), and 208-ft (63.4 m).

Three experiments were performed at the 58-ft (17.7 m) depth, all with the 3-inch (7.62 cm)-diameter pile segment models.

As noted earlier in Chapter 4, certain aspects of the results of the experiments at the 58-ft (17.7 m) depth are clouded by the conditions of installation under which the consolidation was initiated. Since the thin-walled cutting shoes plugged prematurely, the soil pressures against the probes were larger than those which would be created by the insertion of a thin-walled pile, yet were smaller than those created by a fully-plugged pile. Since the lateral pressure measurements were made at the transition depth between the two pressure conditions, some vertical gradients in pressure may have existed, the effects of which are uncertain with regard to the magnitude and the rate of dissipation of the excess pore pressures during radial consolidation.

As will be shown later, the effects of the increased level of lateral pressures on the magnitudes of shear transfer which were developed were not significant; however, the time required for consolidation may have been increased.

The first experiment at the 58-ft (17.7 m) depth consisted of a large-displacement, two-way cyclic test at a high degree of consolidation. For the next two experiments, the probes were load-tested at a lower degree of consolidation, but with differing load histories. In addition, in Experiments 2 and 3, the probes were allowed to rest for varying periods of time after the initial series of load tests, in order to observe the process of recovery of shear transfer capacity by the soil after cyclic degradation.

5.2.1 Experiment 1

Experiment 1 at the 58-ft (17.7 m) depth was performed in order to record the long-term consolidation response of the soil, without any intermediate disturbance caused by cyclic loading at low degrees of consolidation. It was also planned to use the results of this experiment to obtain the value of shear transfer developed after long-term, undisturbed consolidation, so that the results of parallel experiments, with load tests performed at lower degrees of consolidation, could be used to develop a relationship describing the time-rate of development of shear transfer.

For this experiment, a probe was installed on 30 November 1982 at 19:41, and tested to failure in tension and compression 9 minutes after driving. Consolidation was then allowed to proceed for 4323 minutes, at which time a large-displacement, two-way cyclic test was performed.

The variations in the soil pressures during consolidation are shown in Plate 5.1. As shown in the Plate, the maximum total pressure after installation was 10.62 ksf (508 kPa), with a simultaneous maximum pore pressure of 10.57 ksf (506 kPa). The increases in pore and total pressure (and the decrease in radial effective pressure) after 2400 minutes of consolidation occurred while an adjacent borehole was being prepared for the second experiment. The break in the curve between the times of 8 and 17 minutes after driving correspond to the time of the load test performed immediately after driving.

During the first and second experiments, drilling mud was added to the seawater used to flush the cuttings from the boring. During all subsequent experiments, only saltwater gel was added, with no excess weight being used to keep the borehole open, and in order to avoid such disturbance in the future experiments.

At the time the disturbance occurred, the total radial pressure had decreased to 9.37 ksf (448 kPa), with a value of pore pressure of 8.93 ksf (427 kPa), yielding a radial effective pressure of 0.44 ksf (21 kPa). At the time of the load test, the radial total pressure was 9.70 ksf (464 kPa) and the pore pressure was 9.45 ksf (452 kPa), yielding a value of radial effective pressure of 0.25 ksf (12 kPa), smaller than the value prior to the disturbance.

The shear transfer-displacement relationships recorded during the first quarter of the first and the eleventh cycles of the test are shown in Plate 5.2. The maximum shear transfer on the first cycle was 0.13 ksf (6 kPa); the shear transfer reduced to 0.07 ksf (3 kPa) on the eleventh cycle. The peak shear transfer was thus severely reduced by the large-displacement cyclic loading, with the soil retaining only 54 percent of the initial static shear transfer capacity.

During the load test performed 9 minutes after driving, also shown in Plate 5.2, the shear transfer was 0.05 ksf (2 kPa), yielding a ratio of 2.6 for the maximum shear transfer after consolidation as compared with the value observed immediately after driving, with only a minor increase in the cyclic minimum resistance.

The variation in the pore pressure and radial effective pressure with time during the cyclic load test are given in Plate 5.3, which also contains the variation with time in the shear transfer. The delay in the load test was necessary to reset scales on the analog plotter and to adjust the flow control valves on the hydraulic loading system.

As seen in Plate 5.3, the initial loading of the probe to failure resulted in an increase in pore pressure, with an accompanying decrease in the radial effective pressure. During the remainder of the cyclic tests, the radial pressures tended to fluctuate about mean values of pressure which were higher (pore pressure) and lower (effective pressure) than the pre-test values. The pattern of the pressure fluctuations consists of an abrupt decrease in the pore pressure during plastic slip and a return toward an ambient value during the reversal of load during each cycle.

At the conclusion of the cyclic tests, static pressure measurements showed a radial total pressure of 9.67 ksf (463 kPa) (essentially no change) and a pore pressure of 9.52 ksf (456 kPa) (a slight increase).

The pattern of response suggests a dilation of the near-pile clay during plastic slip, with the decrease in pore pressure resulting in flow of water into the shear zone. Such behavior would result in an increase in the void ratio (thereby moisture content), thus decreasing the shear strength, and reducing the shear transfer capacity of the soil.

If the clay in the shear zone has increased in volume due to the inward flow of pore water during plastic slip, the end result at the conclusion of the process should be an increase in the pore pressure in the near-pile clay, since the volume of the cylindrical body inside the surface of slip has been increased, much like cavity expansion. The pore pressures in the clay outside the surface of slip may be little affected, except for slight increases due to the reversal of shear deformation. Thus, the radial gradient in pore pressure at the conclusion of the test should be very steep near the pile, with the excess pore pressure approaching the ambient value a short radial distance away.

Since the probe was removed from the boring shortly after the cyclic load test, any recovery with time in the shear transfer capability of the soil, or any dissipation of the excess pore pressure, were not observed.

5.2.2 Experiment 2

The primary purpose of Experiment 2 at the 58-ft (17.7 m) depth was to obtain a value of shear transfer at an intermediate degree of consolidation. Load tests were also performed after a period of reconsolidation to observe the effects of time and consolidation on the recovery of shear transfer capacity after complete cyclic degradation in resistance at a low degree of consolidation.

For this experiment, a probe was installed on 1 December 1982 at 11:38, tested to failure in tension and compression 11 minutes after driving, and then allowed to consolidate for a total of 418 minutes (7 hours) after driving. The probe was then subjected to ten cycles of large-displacement, two-way loading. Upon completion of the cyclic load test, the soil was allowed to consolidate until 1165 minutes (19.4 hours) had elapsed after installation. The probe was then subjected to one cycle of failure in tension and compression, then removed from the boring.

The variations in the soil pressures during consolidation are shown in Plate 5.4. The breaks in the curves correspond to the times and durations of the load tests, during which the pressure variations are given in separate plates. The maximum total pressure immediately after driving was 11.60 ksf (555 kPa), with a maximum pore pressure of 10.83 ksf (518 kPa). During the 418-minute period of undisturbed consolidation, the total pressure reduced to 10.81 ksf (517 kPa), with a simultaneous reduction in the pore pressure to 9.75 ksf (466 kPa), yielding a nominal increase in the radial effective pressure from 0.77 (37 kPa) to 1.06 ksf (51 kPa).

During a period of reconsolidation following the large-displacement cyclic load test, the total pressure reduced from 10.84 (519) to 10.45 ksf (500 kPa), accompanied by a reduction in the pore pressure from 10.02 (479 kPa) to 9.44 ksf

(452 kPa), yielding an increase in the value of radial effective pressure from 0.82 (39 kPa) to 1.01 ksf (48 kPa).

The shear transfer recorded during the load test performed 11 minutes after driving, shown in Plate 5.5, was 0.05 ksf (2 kPa).

The shear transfer-displacement relationships recorded during the load test after 418 minutes of consolidation are also shown in Plate 5.5. The maximum shear transfer on the first cycle was 0.14 ksf (7 kPa); after ten cycles, the shear transfer was reduced to 0.09 ksf (4 kPa).

Thus, severe losses were again observed in the shear transfer capacity of the soil under large-displacement cyclic loading, with the minimum resistance being only 64 percent of that recorded on the first cycle.

The simultaneous variations in the pore pressure, the radial effective pressure, and the shear transfer during the cyclic tests are shown in Plate 5.6. As seen in the Plate, the initial loading to failure resulted in an increase in the pore pressure, which again did not return to the pre-test value until after the loading had ceased. The patterns of pore pressure during the cyclic test were very similar to those observed in the first experiment; abrupt decreases during plastic slip, with a return toward an ambient pressure (which was greater than the pre-test pressure) during the reversal of loading.

Static pressure measurements made after the conclusion of the load tests indicated a value of 10.84 ksf (519 kPa) for the total pressure, indicating virtually no change. A value of 10.02 ksf (479 kPa) was indicated for the pore pressure, which represents an increase of 0.27 ksf (13 kPa). As noted earlier, the excess pore pressures rapidly dissipated, being 9.89 ksf (473 kPa) after only 25 minutes.

The rapid decay of the excess pore pressures at the conclusion of the tests indicates that the excess pressures existed in only a small volume of soil; had the clay been affected to a large radial distance, the dissipation time would have been much longer.

Shortly after these measurements were made, the generator which provided power to the data acquisition building failed. Power was not restored until the following morning, so that intermediate readings are not available.

After the overnight period of rest, the probe was subjected to one cycle of large-displacement loading, with an initial maximum resistance of 0.19 ksf (9 kPa) being recorded, as shown in Plate 5.7. The probe was then removed from the borehole in order to move to the next depth.

A comparison of the peak shear transfer recorded after reconsolidation to that recorded on the first cycle of the earlier test shows that the soil had fully recovered from the effects of cyclic degradation, and had even increased in shear transfer capacity, from a cyclic minimum resistance during the first test series of 0.09 ksf (4 kPa) to 0.19 ksf (9 kPa), larger than the value of 0.14 ksf (7 kPa) recorded on the first cycle of the primary series of cyclic tests.

The shear-displacement behavior recorded during the first cycle of loading to failure during this test is shown in Plate 5.7, with the simultaneous variations in the shear transfer, the pore pressure, and the radial effective pressure during the complete test given in Plate 5.8. As seen in Plate 5.8, the pore pressure increased (with a simultaneous reduction in the radial effective pressure) during the initial loading, with the pressures then showing only a small decrease during plastic slip. Upon reversing the direction of load, the pore pressure again increased (with an accompanying opposite change in the radial effective pressure), with the pressures again decreasing slightly during plastic slip.

On the second reversal of load direction, the pore pressure increased during the load reversal, then decreased during plastic slip (with corresponding opposite changes in the radial effective pressure).

The probe was removed from the boring immediately after the load test, with no static pressure measurements being made while the probe was in the soil.

5.2.3 Experiment 3

Experiment 3 at the 58-ft (17.7 m) depth was performed in order to observe the value of shear transfer at which cyclic degradation would be initiated when the cyclic loading was performed at small displacements. By comparing the value of peak shear transfer recorded after the application of several cycles of loading at small displacements to the cyclic minimum resistance, and to the peak resistance observed in Experiment 2, it could be determined whether the small-displacement cyclic loading adversely affected the shear transfer capacity of the soil.

After the primary series of cyclic tests, a period of reconsolidation was allowed, and an additional load test performed in order to observe the recovery in shear transfer capacity with time after cyclic degradation at a low degree of consolidation.

For this experiment, a probe was installed on 5 December 1982 at 11:26. The probe was loaded to failure in tension and compression 6 minutes after driving. The soil was then allowed to consolidate for 506 minutes (8.4 hours). The probe was then subjected to a series of progressively-increased reversals of symmetric displacement, ending with a large-displacement two-way cyclic test. The soil was then allowed to continue to consolidate until 1150 minutes (19 hours) had elapsed since installation. The probe was then subjected to three cycles of large-displacement loading, before being removed from the borehole.

The variations in the soil pressures during the periods of consolidation between load tests are shown in Plate 5.9. The breaks in the curves correspond to the times of the load tests. As shown in the Plate, the maximum total pressure recorded after installation was 11.80 ksf (565 kPa), with a corresponding maximum pore pressure of 10.97 ksf (525 kPa). After the initial 506 minutes of consolidation, the total pressure had decreased to 10.91 ksf (522 kPa), with an accompanying decrease in the pore pressure to 10.04 ksf (480 kPa), yielding a value of 0.87 ksf (42 kPa) for the radial effective pressure.

As noted in Chapter 3, the fluctuations in the values of pressure are real responses to the vertical movement of the platform structure, a portion of which were transmitted to the probe, and do not represent noise or uncertainty in the measurements.

The maximum shear transfer recorded during the load test performed immediately after driving was 0.12 ksf (6 kPa), which reduced to 0.08 ksf (4 kPa) after one reversal of loading.

During the sequence of progressively-increased symmetric displacement-controlled loadings, the displacements were increased from 0.005 in. (0.13 mm) to 0.030 inch (0.76 mm), followed by several cycles of loading to ±0.050 inch (1.3 mm). The maximum shear transfer, recorded on the initial displacement to 0.020 inch (0.51 mm), was 0.22 ksf (11 kPa). After five cycles of reversal at ±0.050 in. (0.51 mm), the shear transfer reduced to 0.17 ksf (8 kPa).

The shear-displacement response of the probe during the first and last cycle of loading at each set of increased displacement limits are shown in Plate 5.10. The six figures in Plate 5.10 are taken from the tests at ±0.005 (0.13), ±0.010 (0.25), ±0.015 (0.39), ±0.020 (0.51), ±0.025 (0.64), and ±0.030 inch (0.76 mm). The transition from quasi-elastic to almost fully plastic behavior is readily apparent in the plate, with plastic deformation of the soil beginning at very small values of relative pile-soil displacement.

The simultaneous variation with time in the pore pressure, radial effective pressure, and the shear transfer are given in Plate 5.11. The trends shown in the Plate are clear. The average pore pressure tends to increase during the early portion of the cyclic loading, accompanied by a decrease in the radial effective pressure; as more plastic slip is created by increasing the displacement limits, abrupt decreases again occur during plastic slip, with a decrease in the average value.

A cycle-by-cycle comparison of the peak values of shear transfer shows that, indeed, cyclic degradation in the shear transfer capacity was occurring, with small losses occurring within each set of increased sets of displacement limits.

An examination of the pore pressure variations indicate that decreases in pressure accompany the application of shear. If inward flow of pore water toward the pile is the mechanism of cyclic degradation, such response would be expected.

The losses in resistance can also be seen in Plate 5.10; however, at the smaller limits of displacement (with steeper slopes), a part of the apparent loss in resistance is due to slight variations in the end-point displacement.

The shear-displacement response of the probe during the tensile loading to failure of the first cycle of loading between displacement limits of ±0.050 inch (1.3 mm) is shown in Plate 5.12, along with the results of the test performed immediately after driving. As can be seen in Plate 5.11, only minor losses in resistance were observed during the large-displacement cyclic test, therefore only the first cycle is shown.

The absence of any apparent cyclic degradation during the series of cyclic tests does not indicate that none occurred, but merely that the progressive increase in the limits of displacement resulted in losses in resistance at values of shear transfer lower than the value which would have been attained had the probe been immediately loaded to failure.

A comparison of the results of these tests with those from Experiment 2 are inconclusive; the cyclic minimum resistance observed in this experiment, 0.17 ksf (8 kPa), is almost twice that observed in Experiment 2 (Plate 5.5), 0.09 ksf (4 kPa), and is even larger than the peak value of 0.14 ksf (7 kPa) recorded in the earlier experiment.

A comparison of the cyclic minimum shear transfer, 0.17 ksf (8 kPa), to the maximum value recorded during the small-displacement cyclic tests, 0.22 ksf (11 kPa) (on the first cycle of loading to 0.020 inch (0.51 mm)), indicates a loss of 23 percent, as compared with losses of 36 percent in Experiment 2; this suggests that the small-displacement cyclic loading resulted in a loss of approximately 13 percent of the resistance at relative displacements of .005 (0.13), 0.010 (0.25), and 0.015 inch (0.38 mm) (assuming the same ratio of peak to cyclic minimum resistance in both experiments).

A set of static pressure measurements made at the conclusion of the cyclic load tests showed a value of 10.52 ksf (504 kPa) for the total pressure, a reduction of 0.39 ksf (19 kPa) from the pre-test value of 10.91 ksf (522 kPa). The pore pressure at the end of the test was 10.14 ksf (485 kPa), an increase of 0.10 ksf (5 kPa) above the pre-test static value.

The probe was then allowed to remain undisturbed until a total of 1150 minutes had elapsed since the end of driving. At this time, the total pressure was approximately 10.51 ksf (503 kPa) and the pore pressure was 9.67 ksf (463 kPa), resulting in a value of radial effective pressure of 0.83 ksf (40 kPa). Again, the motion of the structure due to wave loading was partially transmitted to the probe, causing the soil pressures to fluctuate in a cyclic manner during wave loading.

As seen in Plate 5.9, the excess pore pressure rapidly returned to the initial consolidation path after the end of the test. This behavior again suggests that the shear-generated pore pressures were confined to a small zone of clay around the pile, forming a localized bump in the radial gradient of excess pore pressure created by the insertion of the probe. After the localized excess pore pressures dissipated, the radial gradient of pore pressure returned to the same pattern as had existed prior to the test, and the consolidation of the larger volume of disturbed clay continued along the path which had been developed during undisturbed consolidation.

The effects of this pattern of consolidation behavior on the development of axial pile capacity can be seen in the values of shear transfer which were recorded at the end of the period of reconsolidation. Since the soil near the shear zone experienced an additional increment of volume change, which accompanied the dissipation of the localized bump in the radial gradient of pressure, the clay in, and very near, the shear zone should have had a lower void ratio than the clay outside the shear zone, and consequently a higher value of shearing resistance.

At the end of the period of reconsolidation, the probe was subjected to three cycles of loading to failure at large displacements, and was finally removed from the borehole. The maximum shear transfer on the first loading to failure in tension (not shown) was 0.22 ksf (11 kPa). This value compares favorably with the maximum value of 0.22 ksf (11 kPa) recorded during the earlier cyclic tests, and represents an increase of 40 percent above the cyclic minimum shear transfer observed in the earlier tests.

The behavior which was observed confirms the concept described earlier. The clay within the initial shear zone which developed during the first series of cyclic tests exhibited a shearing resistance of only 0.17 ksf (8 kPa) at the end of the first cyclic load test. In order for the shear transfer to equal or exceed the level measured on the first cycle of the preceding test, the clay within the shear zone must have increased in strength to a level at least equal to that which existed prior to the initial load test.

If the shear strength of the clay within the shear zone is equal to that which existed previously, then failure in the soil will occur at the same radial distance from the pile wall, since the radial shear stress gradient is the same. If, however, the clay in and near the shear zone increases in shear strength to a value slightly greater than that which previously existed, then the failure surface will be outside the initial shear zone; as a consequence, the shear transfer capacity will be at least equal to, or slightly greater than, that which was available during the initial loading of the pile. In this experiment, the value of shear transfer measured during the retest was essentially equal to that which was measured earlier; however, the value may still be less than the maximum value which would have been observed in the earlier test, had the first cycle of loading been carried to failure.

During plastic slip on the third cycle of loading, shown in Plate 5.13, the rate of displacement was increased from 0.001 inch (0.025 mm)/sec to 0.010 inch (0.25 mm)/sec. The corresponding increase in shear transfer was from 0.20 ksf (10 kPa) to 0.23 ksf (11 kPa), an increase of 15 percent for the ten-fold increase in slip rate.

The simultaneous variations in the shear transfer, pore pressure, and radial effective pressure during these load tests are shown in Plate 5.14. As shown in the Plate, the behavior differed from that which was observed previously, with the pore pressures generally remaining lower than the pre-test values and the radial effective pressures above the pre-test values. The effects of plastic slip in the shear transfer on the soil pressures, however, were the same; the soil behavior is dilative, with pore pressures decreasing during plastic slip, accompanied by the increases in the radial effective pressure. During the reversal of loading, the pressures tend to return to the pre-test static values. Upon the enforcement of plastic slip, the pore pressures again rapidly decrease.

The magnitudes of the increases in the radial effective pressure are quite large when compared to the static values recorded prior to the load test, and to the magnitudes of shear transfer. Prior to the beginning of the test, the static radial effective pressure was 0.83 ksf (40 kPa). During the second failure in compression, the radial effective pressure increased from a value of 0.61 ksf (29 kPa) at yield to a maximum of 1.81 ksf (87 kPa), with a decrease in the shear transfer, from a maximum of 0.26 ksf (12 kPa) at yield to 0.23 ksf (11 kPa) during plastic slip.

Such behavior should be expected; as the shear zone dilates, the pore pressure decreases. Since the total pressure acting against the shear surface changes only slightly (indicating no significant volume change is occurring), a large increase in the calculated value of radial effective pressure results.

At the conclusion of the load test, the ram was extended to its full capacity to break the cutting shoe free, and the probe was removed. No static pressure measurements were made following the load tests with the probe in the soil.

5.2.4 Summary of Results at the 58-ft (17.7 m) Depth

The results of the load tests performed during the experiments at the 58-ft (17.7 m) depth are tabulated below. The values of soil pressure given in the table are those measured under quiescent (static) conditions before and after the load tests. Because of the radial gradients in pore pressure during active shearing, the transient values recorded during the load tests are not accurate representations of the actual pressure; thus, the values are not shown. The values given in the table are representative of the actual pressures existing in the soil adjacent to the pile; any effects of cyclic loading and degradation on the radial pressure conditions can be determined from the changes in the static pressures measured before and after the cyclic tests.

TABLE 5.1 RESULTS OF THE EXPERIMENTS AT THE 58-FT (17.7 M) DEPTH

Experiment Number	Date	Time	Elapsed Time (min)	Soil Pressures Total ksf (kPa)	Soil Pressures Pore ksf (kPa)	Shear Transfer / Cycle Num ksf (kPa)
1	30 Nov 1982	19:50	9	10.55 (505)	10.54 (504)	0.05 (2)
	03 Dec 1982	19:44	4323	9.70 (464)	9.45 (452)	0.13 / 1 (6)
		20:33	4372	9.67 (463)	9.52 (456)	0.07 / 11 (3)
2	01 Dec 1982	11:49	11	11.53 (552)	10.49 (502)	0.05 (2)
		19:35	418	10.81 (517)	9.75 (467)	0.14 / 1 (7)
		20:19	462	10.84 (519)	10.02 (479)	0.09 / 10 (4)
	02 Dec 1982	07:03	1165	10.45 (500)	9.44 (452)	0.19 (9)
3	05 Dec 1982	11:32	6	11.49 (550)	10.94 (523)	0.12 (6)
		19:52	506	10.91 (522)	10.04 (480)	N/A
		23:29	717	10.52 (503)	10.14 (485)	0.17 / 40 (8)
	06 Dec 1982	06:36	1150	10.51 (503)	9.67 (463)	0.22 (11)

The lack of agreement among the values of maximum and cyclic minimum shear transfer among the three experiments precludes the drawing of any general conclusions regarding the behavior of the soil at this depth, except that, during each experiment, losses of up to 46 percent of the initial static capacity were observed. Although, in Experiments 2 and 3, the losses were shown to be of a temporary nature, the momentary capacity during the most severe loading conditions (when higher capacities are needed) would be reduced to a fraction of the static capacity.

Thus, for static pile design, the higher values may be used; however, since the design loads occur during storms, with the loading being cyclic in nature, the potential loss of up to 46 percent of the static capacity must be considered.

The lack of reproducibility in the magnitudes of shear transfer in the experiments was undoubtedly aggravated by the effects of premature plugging on the consolidation behavior and, thereby, on the time rate of development of shear transfer; however, the magnitudes of the shear transfer were very small, with very low lateral pressures; perhaps better reproducibility should not be expected.

As seen in the table, the effects of the disturbances during drilling and the use of weighted drilling fluid on the pressures measured in Experiment 1 (See Plate 5.1) adversely affected the process of consolidation during this experiment, especially the total pressure and the shear transfer, with final values of radial effective pressure and shear transfer which differ considerably from those observed in Experiments 2 and 3.

The agreement among the values of pressure for the three experiments (with the exception of the total pressure from Experiment 1), however, indicates very similar behavior during consolidation.

The results of these experiments have been included in this report primarily for completeness. The resistance to axial pile loading in shallow, soft soils is relatively unimportant, since only a small percentage of the total capacity of a pile is developed at such depths. The results of the experiments at the 148 (45.1), 178 (54.3), and 208-ft (63.4 m) depths are of much more significance in relation to interpretation of the behavior of the large-diameter instrumented pile, and to axial piles in general; thus, the efforts have been concentrated toward the experiments performed at the greater depths.

5.3 The Experiments at the 148-ft (45.1 m) Depth

The experiments at the 148-ft (45.1 m) depth were the only experiments performed in the Stratum II soil, which is approximately bounded by depths of 80 (24.4) and 160 feet (48.8 m) below the mudline.

A total of five experiments were performed at this depth.

Three of the experiments were performed under CNRD Project 13-2 in 1982 with the 3-inch (7.62 cm)-diameter pile segment models equipped with the thin-wall cutting shoes.

Two experiments were added to those planned in 1983 under CNRD 13-3, one with a 3-inch (7.62 cm)-diameter model with a thin-wall cutting shoe, and one with the 1.72-inch (4.37 cm)-diameter pile segment model, the X-probe, which acts as a full-displacement pile.

One additional experiment was initiated during the field work in 1983; however, the cutting shoe encountered a scrap piece of wire rope which had fallen into the borehole. Extremely hard driving was encountered, yet only hydrostatic pressures were measured, with very low values of shear transfer being measured during the immediate load test. Upon removal of the probe, the tip of the cutting shoe was found to be dented, with the indentations bearing the marks of the wire rope. While drilling to the next depth, the piece of wire rope was also encountered by the drill bit. The wire rope was not recovered, and became either pressed into the wall of the borehole or wrapped around the drill string. It was not encountered during the later experiments. Since the drill string was left in the boring to serve as a casing, the fate of the piece of wire rope is not known.

During these experiments, a wide range in undisturbed consolidation time prior to load testing the probes was allowed, so as to observe the increase in shear transfer with time (Experiments 1, 3, and 4). One experiment was performed to duplicate the installation conditions at the 58-ft (17.7 m) depth (Experiment 2), while one experiment was performed with the 1.72-inch (4.37 cm)-diameter, full-displacement X-probe (Experiment 5). The experiments also included load tests performed with various periods of reconsolidation after the initial series of cyclic tests, in order to observe the process of recovery in shear transfer capacity (Experiments 3, 4, and 5).

5.3.1 Experiment 1

Experiment 1 at the 148-ft (45.1 m) depth was performed to observe the long-term consolidation response of the soil, without any disturbance due to load testing at intermediate stages of consolidation. The long-term shear transfer obtained during the experiment could later be compared with that from the experiments performed in parallel borings at this depth to observe the development of shear transfer capacity with time.

For this experiment, a probe was installed on 4 December 1982 at 17:18. A load test was performed 11 minutes after driving. The soil was then allowed to consolidate for 4081 minutes. Ten cycles of large-displacement cyclic loading were then applied, after which the probe was removed from the boring.

The variations in the soil pressures during consolidation are shown in Plate 5.15. The break in the curves in Plate 5.15 corresponds to the time at which the load test was performed, between 10 and 20 minutes after driving. Immediately after driving, the total radial pressure was 18.38 ksf (879 kPa), with a pore pressure of 18.33 ksf (877 kPa). As shown in Plate 5.15, both the total and the pore pressure continued to increase with time, with values of 18.60 ksf (890 kPa) and 18.30 ksf (876 kPa) being recorded after the completion of the immediate load test. The total and pore pressures then began to decrease, reaching values of 18.21 ksf (871 kPa) and 17.09 ksf (818 kPa) at the end of the period of consolidation, indicating an increase in the radial effective pressure from 0.30 ksf (14 kPa) to 1.13 ksf (54 kPa).

The shear transfer-displacement behavior recorded during the test immediately after driving is given in Plate 5.16. The limiting shear transfer after one full reversal of plastic slip was 0.15 ksf (7 kPa).

The results of the large-displacement cyclic loading are also shown in Plate 5.16, which contains the first quarter of first and the tenth cycles of loading. The maximum shear transfer on the first cycle was 0.36 ksf (17 kPa). After ten cycles of loading, the shear transfer had been reduced to 0.24 ksf (11 kPa), a reduction of 33 percent.

The simultaneous variations in the pore pressure, the radial effective pressure, and the shear transfer during the cyclic tests are shown in Plate 5.17. As shown in the Plate, the pore pressure increased during the initial loading to failure, with a simultaneous decrease in the radial effective pressure. On subsequent cycles of loading, the pore pressure decreased during plastic slip, with a simultaneous increase in the radial effective pressure. The behavior was very similar to that observed in the experiments at the 58-ft (17.7 m) depth, with abrupt decreases in the pore pressure accompanying plastic slip.

It should be noted that, in all the experiments, failure occurred on a soil-soil surface, and not at the pile wall. This was clearly proven by the adhesion of a layer of clay on the surface of the probes after removal of the probes from the boring. Since failure occurred in the soil near the pile, and not at the pile-soil interface, the meaning of the pressures which were recorded during active loading must be reviewed.

In general, two pore pressure effects were observed to accompany the cyclic load tests. First, static pressure measurements made at the end of the load tests indicated that the cyclic loading had resulted in the creation of excess pore pressures, which rapidly dissipated upon the cessation of loading. Secondly, both analog and digital records of the pore pressures during the active loading of the probes indicated wide fluctuations in the pressure, which were predominantly due to decreases in the pore pressure during plastic slip.

In order to relate the effects of the variations in the pore pressure during loading to the shear transfer which is developed along a pile, and the relationships between the pore pressures, cyclic degradation, and recovery in resistance during reconsolidation, each effect must be considered separately.

The rapid decrease in pore pressure which accompanies plastic slip, and the equally-rapid return toward the ambient pressure during the reversal of loading, suggests that the pressure fluctuations are occurring in a thin circumferential band of clay. If the layer involved is thin, then the local movement of only a small volume of pore water is required to relieve the pressure deficiency. Since the pore pressure changes during plastic slip are negative, a temporary gradient in the radial variation in pore pressure must exist, with the lowest value of pressure being in the thin zone of concentrated slip. Thus, pore water is drawn into the shear zone, resulting in an increase in the moisture content, and a decrease in the strength of the soil. The effect may be described as a lubrication of the failure surface, with the end result being the formation of a thin circumferential band of clay having a higher void ratio and a lower shear strength than the surrounding soil.

The direction of the changes in pore pressure which accompany the plastic slip tend to support this concept. As the shear zone dilates, or expands, the pore pressures within the shear zone rapidly decrease. The depression in the pore pressure in the zone of shear draws in pore water from the adjacent clay, increasing the moisture content, resulting in a loss of shear strength; thus a degradation in the shear transfer capacity.

The increase in the volume of the clay within the shear zone results in an increase in the pore pressure in the clay between the pile wall and the failure surface. These excess pressures take longer to dissipate than those which result from dilation of the thin shear zone.

There are thus two simultaneous pore pressure effects which are superimposed on each other: 1) a general increase in the pore pressures in a circumferential layer of clay near the pile, due to increases in the volume of the soil contained within the failure surface, and 2) highly-localized depressions of the pore pressure in a thin layer of clay adjacent to the failure surface during active plastic slip.

If the concept described above is correct, then attention must be given to the values of pressure which are recorded during the cyclic load tests.

If failure and plastic slip occur in the soil near the pile, and not at the pile wall, then a pressure gradient must exist in the clay which lies between the pile wall and the shear zone. If a gradient exists, then the pore pressures which are measured at the pile wall are not those which really exist in the shear zone. The values of pore pressure which are recorded during active loading are thus merely indicators of volume changes that are occurring in the shear zone, and not the exact magnitudes of pore pressure which exist on the slip surface.

The values of static pore pressure which are measured during consolidation, and during periods of inactivity, are true measures of the pore pressures which exist in the soil. However, those values recorded during active loading, when the rapid fluctuations in pressure occur, do not exactly represent the actual pore pressures in the soil.

The pattern of consolidation and dissipation of excess pore pressures after the end of the cyclic tests tends to support this concept; i.e., rapid dissipation of the excess pore pressures at the end of the tests, followed by a return to the pattern established during undisturbed consolidation.

Prior to removing the probe, one additional cycle of loading was performed, with the results shown in Plate 5.18. During this test, the rate of displacement was varied during plastic slip. As shown in the plate, a value of shear transfer of 0.27 ksf (13 kPa) was recorded at a slip rate of 0.001 inch (0.025 mm)/sec, with an increase in the shear transfer to 0.31 ksf (15 kPa) at a slip rate of 0.012 inch (0.31 mm)/sec. The rate of displacement was then decreased, with the shear transfer subsequently returning to a smaller value.

The simultaneous variations in the pore pressure, the radial effective pressure, and the shear transfer during the variable-rate test are shown in Plate 5.19. As shown in the plate, the increase in the rate of slip resulted in an increase in the magnitudes of the changes in the radial soil pressures. During the last loading to failure in tension (near the 6-minute mark) the radial effective pressure increased from a value of 0.73 ksf (35 kPa) at a slip rate of 0.001 inch (0.025 mm)/sec to 1.52 ksf (73 kPa) at a slip rate of 0.100 inch (2.5 mm)/sec, with the corresponding values of shear transfer given above.

Upon completion of this loading, the probe was removed from the boring, with no static pressure measurements being made while the probe was in the soil.5.3.2

5.3.2 Experiment 2

For Experiment 2 at the 148-ft (45.1 m) depth, the drilling procedure which had been used in the experiments at the 58-ft (17.7 m) depth was followed. This was premature plugging, the results of which could be used to estimate the effects on the results at the shallower depth. Only two series of load tests were performed; the results of which can be compared with tests performed during Experiment 3 at similar times after driving, in order to better evaluate the results of the experiments at the 58-ft (17.7 m) depth.

For this experiment, a probe was installed on 2 December 1982 at 14:10. A cycle of loading to failure in tension and compression was performed 8 minutes after installation. The soil was then allowed to consolidate for 514 minutes, at which time a large-displacement two-way cyclic test was performed. Upon completion of the load test, the consolidation was allowed to continue until 1013 minutes had elapsed, at which time one cycle of reversed large-displacement loading was applied, and the probe removed.

The variations in soil pressure during consolidation are shown in Plate 5.20. Again, the breaks in the curves correspond to the times of the load tests. As shown in the Plate, the total pressure immediately after driving was 20.34 ksf (973 kPa), with the pore pressure being 19.50 ksf (933 kPa).

At the end of the immediate load test, the total pressure was 20.23 ksf (968 kPa), with a pore pressure of 19.76 ksf (946 kPa). After 485 minutes of undisturbed consolidation, the total pressure was 19.83 ksf (949 kPa); the pore pressure was 18.15 ksf (868 kPa), yielding a value of radial effective pressure of 1.67 ksf (80 kPa). Immediately prior to the final load test after 1010 minutes of consolidation, the total pressure had decreased to 19.30 ksf (924 kPa) and the pore pressure had decreased to 17.34 ksf (830 kPa), yielding a value of 1.97 ksf (94 kPa) for the radial effective pressure.

During the load test performed immediately after driving, the shear transfer was 0.24 ksf (11 kPa) on the initial loading, which reduced to 0.14 ksf (7 kPa) after one cycle of reversal. The portion of the load test recorded following the reversal of loading is shown in Plate 5.21.

The results of the first quarter of the first and the twelfth cycles of loading to large reversed displacements after 514 minutes of consolidation are also given in Plate 5.21. The maximum shear transfer on the first cycle was 0.41 ksf (20 kPa). After twelve cycles, the shear transfer was reduced to 0.30 ksf (14 kPa), a loss of 27 percent of the static capacity, approximately the same percentage as was observed in the first experiment.

The simultaneous variations in the pore pressure, the radial effective pressure, and the shear transfer during the cyclic tests are shown in Plate 5.22. As seen in the Plate, the digital recording system was turned off during much of the test, in order to obtain better analog recordings of the shear transfer-displacement behavior. One cycle of loading is also shown in Plate 3.5.

As seen in Plate 5.22, the initial loading to failure in tension on the first cycle resulted in a simultaneous decrease in the radial effective pressure and an increase in the pore pressure. Again, the changes in the radial effective pressure are not due to pore pressure effects alone, as can also be seen in Plate 5.20, which shows the decreased static total pressure at the end of the cyclic test. The decreases in pore pressure again accompany plastic slip.

The static pressures measured after the end of the cyclic tests indicated a value of 19.65 ksf (940 kPa) for the total pressure, a reduction of 0.17 ksf (8 kPa), and a value of 18.29 ksf (875 kPa) for the pore pressure, an increase of 0.14 ksf (7 kPa), resulting in a decrease of 0.31 ksf (15 kPa) in the radial effective pressure, to a value of 1.36 ksf (65 kPa).

After the consolidation had been allowed to continue for a period of 1013 minutes since installation, the probe was subjected to one cycle of loading to large reversed displacements. On the initial loading to failure in tension (not shown) the maximum shear transfer was 0.42 ksf (20 kPa), indicating that the static capacity had been fully recovered.

The simultaneous variation in the pore pressure, the radial effective pressure, and the shear transfer during the load test are shown in Plate 5.23. As seen in the plate, the pore pressure increases during the initial loading of the probe, until plastic slip occurs in the soil. At this point, the pore pressure decreases dramatically, with a simultaneous, but slightly smaller, increase in the calculated value of radial effective pressure.

The probe was then subjected to one cycle of reversal of plastic slip. During each post-yield plastic slip, the pore pressure decreased, increasing during the reversal of load; corresponding opposite changes may be noted in the radial effective pressure.

During the last pull upward on the probe, the rate of travel was increased from 0.001 inch (0.025 mm)/sec to the fastest rate possible (the LVDT had reached a mechanical stop; the absence of displacement measurements precluded establishing a slip rate). Due to the increase in slip rate, the shear transfer increased from 0.39 ksf (19 kPa) to 0.62 ksf (30 kPa); an increase of 0.23 ksf (11 kPa), or 55 percent. Simultaneously, the pore pressure decreased from 17.2 ksf (823 kPa) to 16.5 ksf (790 kPa), with the radial effective pressure increasing from 2.0 (96) to 2.8 ksf (134 kPa).

The hydraulic ram was then extended to its full upward travel to free the cutting shoe, and the probe was removed from the boring.

The behavior observed in this experiment tends to reinforce the concepts of axial pile-soil interaction outlined earlier; once failure and plastic slip occur in a thin circumferential band of clay, the temporary, degraded, resistance which is available during cyclic loading, and the resistance which is available after the shear-generated excess pore pressures dissipate, are determined by changes in void ratio and moisture content which occur in the shear zone.

The cyclic loading was performed in essentially undrained conditions; thus, the shear strength of the clay should remain essentially constant, and should be independent of the magnitude of the lateral pressure. Except for structural changes which occur in the soil during the reversal of plastic shearing, the effects of cyclic degradation cannot be occurring in a large volume of soil. In order for changes in void ratio to be responsible for the losses in shear strength, the volume of clay which is affected must be small, due to the time required for flow of pore water to occur.

Since the losses in resistance occur rapidly, the flow path for the pore water must be short. The abrupt decreases in pore pressure during plastic slip indicate that a steep gradient of pore pressure is formed in the clay adjacent to the shear zone, and that pore water is drawn into the shear zone, increasing the void ratio and the moisture content of the clay within the shear zone, and thus reducing the available shearing resistance.

The recovery in shear transfer capacity with time after the end of the load tests also suggests that the volume of clay that had been affected by cyclic degradation is small. In order for the shear strength of the clay to increase as rapidly as was observed, the changes in void ratio must be associated with the dissipation of the excess pore pressures which arise from the increase in volume of the clay within the shear zone, and not with the dissipation of the excess pore pressures which were created by insertion of the pile.

Again, this mechanism of recovery requires that only a thin circumferential band of clay be involved, and that the dissipation of a localized peak, or bump, in the radial gradient of pore pressure results in a decrease in the void ratio of the clay within the shear zone to a value equal to, or smaller than, that of the clay in the adjacent soil mass.

5.3.3 Experiment 3

The primary purpose of Experiment 3 at the 148-ft (45.1 m) depth was to obtain a value of peak shear transfer at a low degree of consolidation which can be compared with that measured during Experiment 1 to observe the increase in shear transfer with time during undisturbed consolidation. In addition, two series of cyclic load tests were performed at later times, in order to observe the effects of reconsolidation and time on the recovery of shear transfer capacity after cyclic degradation at two intermediate stages of consolidation.

The results of the first two series of cyclic tests can also be compared with load tests performed at similar times after driving during Experiment 2 at this depth, in order to evaluate the effects of premature plugging on the development of shear transfer capacity with time, and to estimate the effects of similar initial conditions on the results of the experiments at the 58-ft (17.7 m) depth.

For this experiment, a probe was installed on 6 December 1982 at 11:46. Two cycles of reversed large-displacement loading were applied 6 minutes after the probe was installed, with large-displacement, two-way cyclic tests performed after 501, 1228, and 2638 minutes of consolidation.

The variations of soil pressure during consolidation are shown in Plate 5.24. One minute after insertion, the total pressure was 18.57 ksf (889 kPa), and the pore pressure was 18.13 ksf (868 kPa), yielding a value of 0.44 ksf (21 kPa) for the radial effective pressure. The breaks in the curves correspond to the times and durations of the load tests; the pressure variations during the cyclic tests are contained in additional plates.

After 501 minutes of consolidation, the total pressure was 18.40 ksf (880 kPa), and the pore pressure had reduced to 17.54 ksf (839 kPa), yielding a value of radial effective pressure of 0.86 ksf (41 kPa).

After 1228 minutes, the total pressure was 18.13 ksf (868 kPa), and the pore pressure was 17.21 ksf (823 kPa), yielding a value of 0.92 ksf (44 kPa) for the radial effective pressure.

After 2638 minutes, the total pressure was 18.17 ksf (869 kPa), and the pore pressure was 17.00 ksf (813 kPa), yielding a value of 1.17 ksf (56 kPa) for the radial effective pressure.

The shear transfer recorded 6 minutes after installation was 0.24 ksf (11 kPa) on the initial loading to failure in tension, 0.14 ksf (7 kPa) on the second, and 0.10 ksf (5 kPa) on the third. Plate 5.25 contains the results of the third loading to failure in tension.

The results of the load tests performed after 501 minutes of consolidation are also shown in Plate 5.25. The Plate contains the shear transfer-displacement curves recorded on the first quarter of the first and the twelfth cycles of reversed large-displacement cycling. The maximum shear transfer on the first cycle was 0.35 ksf (17 kPa), which was reduced to 0.22 ksf (11 kPa) on the twelfth cycle. The loss in shear transfer capacity during these tests was 37 percent, somewhat higher than the losses observed in the first two experiments.

The simultaneous variations in the pore pressure, the radial effective pressure, and the shear transfer during the cyclic tests are shown in Plate 5.26. As seen in the figure, significant variations in the radial pressures accompanied the plastic slip. At near-constant values of shear, the pore pressures decreased by as much as 1.1 ksf (53 kPa), with a corresponding increase in the radial effective pressure.

Static pressure measurements made immediately upon the conclusion of the cyclic test showed a value of 18.33 ksf (877 kPa) for the total pressure and a value of 17.28 ksf (827 kPa) for the pore pressure. For the next two minutes, the pore pressure increased, reaching a maximum value of 17.72 ksf (848 kPa); the total pressure decreased slightly, to 18.22 ksf (872 kPa).

This pattern of pore pressure response can be seen to be related to the flow of pore water in the near-pile clay, as follows.

Due to the increase in volume of the clay within the shear zone (resulting in an increased moisture content), a local increase of pore pressure, from 17.54 ksf (839 kPa) to 17.72 ksf (848 kPa) (or higher), was created.

In addition to the increase in pore pressure in the clay between the pile wall and the failure surface, a localized depression in the pore pressure within a thin layer of clay adjacent to the failure surface resulted in a decrease in the pore pressure measured at the pile wall, with a value of 17.28 ksf (827 kPa) being recorded at the end of loading. As seen in Plate 5.26, the pore pressures were temporarily much lower during plastic slip, with a minimum value of 16.66 ksf (797 kPa) being recorded.

After the end of the load test, as pore water flowed into the shear zone, the pressure gradient toward the shear zone (outward from the pile wall, inward from the adjacent soil) was relieved. Once the pressure deficiency within the shear zone had been relieved, outward flow of pore water from the localized bump in the radial pore pressure gradient began.

The concept of only a thin band of clay being involved in the process of cyclic degradation is thus reinforced, by examining the patterns of pore pressure response, and thereby deducing the direction of flow that must have occurred. Had the depression in pore pressure occurred in a large volume of clay, the time required for the pressure decrease to be relieved would have been considerably longer, since a larger volume of pore water (and a longer flow path) would have been required.

As seen in Plate 5.24, the localized excess pore pressures again dissipated quickly, approaching the value measured prior to the test in only 30 minutes. Once the localized excess pore pressures dissipated, the consolidation path then returned to that established earlier, indicating that the excess pore pressures caused by the insertion of the probe were also dissipating.

At the end of the period allowed for reconsolidation, the total pressure had decreased to 18.13 ksf (868 kPa) and the pore pressure had decreased to 17.21 ksf (823 kPa), combining to result in an increase in the radial effective pressure to 0.92 ksf (44 kPa).

Selected results from the load tests performed after 1228 minutes of consolidation are shown in Plate 5.27. Ten cycles of reversed large-displacement loadings were applied, with a value of maximum shear transfer of 0.39 ksf (19 kPa) being recorded on the first cycle, and a minimum value of 0.25 ksf (12 kPa) on the tenth cycle. The reduction in shear transfer capacity due to cyclic degradation was 36 percent, very close to the degree of loss observed during the first cyclic load test.

A comparison of the value of peak shear transfer recorded on the first cycle of this series of tests with that recorded during the first series of cyclic tests indicates that the soil had fully recovered from the earlier cyclic degradation. The peak shear transfer, 0.39 ksf (19 kPa), is significantly larger than the cyclic minimum resistance of 0.22 ksf (11 kPa) recorded during the earlier cyclic test, and is also slightly larger than the value of 0.35 ksf (17 kPa) recorded on the first cycle of the earlier tests.

The variations in the measured pore pressure, the calculated radial effective pressure, and the shear transfer during the cyclic test are shown in Plate 5.28. During this series of cyclic tests, the pore pressure fluctuated about a mean value which was lower than the value recorded prior to loading, again suggesting the inward flow of pore water toward the pile.

Static pressure measurements made at the conclusion of the load tests showed a value of 17.98 ksf (860 kPa) for the total pressure, a decrease of 0.15 ksf (7 kPa), and an increase of 0.18 ksf (9 kPa) in the pore pressure, to a value of 17.39 ksf (832 kPa). The combined effects yield a reduction of 0.34 ksf (16 kPa) in the radial effective pressure, from 0.92 (44) to 0.58 ksf (28 kPa).

Following this series of cyclic tests, the probe was left in place until 2638 minutes had elapsed after installation. The probe was then subjected to six cycles of large-displacement cyclic loading, then removed from the boring. During this period, the total pressure fluctuated due to the motion of the platform, with a value of 18.17 ksf (869 kPa) being recorded just prior to the final series of load tests. The pore pressures also fluctuated slightly, with a value of 17.00 ksf (813 kPa) being recorded prior to the load tests, yielding a value of 1.17 ksf (56 kPa) for the radial effective pressure.

The results of portions of the load tests are shown in Plate 5.29, which contains the first quarter-cycle from cycles one and six. The maximum shear transfer on the first cycle was 0.30 ksf (14 kPa). After six cycles, the shear transfer was 0.26 ksf (12 kPa), practically the same value that had been measured during the earlier tests.

The increase in the maximum shear transfer from 0.25 ksf (12 kPa) on the last cycle of the preceding test to 0.30 ksf (14 kPa) on the first cycle of this test indicates that the soil within the shear zone had only partially recovered from the effects of cyclic degradation, with the recovery of only one-third of the static capacity which was lost during the earlier cyclic test. The value of the cyclic minimum shear transfer, 0.26 ksf (12 kPa), suggests that failure occurred in the soil on the same shear zone which had been developed in the prior cyclic tests, with no outward shift of the failure surface.

The variations in the measured shear transfer and pore pressure, and the calculated radial effective pressure, are shown in Plate 5.30. As noted in the preceding tests (Plate 5.28), the pore pressures decreased during the initial loading and remained depressed throughout the tests, with corresponding increases in the radial effective pressure. It may be noted that, again, the abrupt decreases in pore pressure (and increases in the radial effective pressure) occur during plastic slip, with near-constant values of limiting shear transfer.

Upon completion of six load cycles, the rate of loading was increased, and one complete cycle of reversed loading was performed. The pressure fluctuations during this cycle are also included in Plate 5.30. The tension-loading portions of the tests are shown in Plate 5.31. As seen in the plate, the effect of the increase in slip rate from 0.001 inch (0.025 mm)/sec to 0.012 inch (0.30 mm)/sec resulted in an increase in the limiting shear transfer, from 0.23 (11) to 0.27 ksf (13 kPa).

At the conclusion of this test, one additional cycle of loading was performed, in which the rate of displacement was increased during plastic slip. During this test, the load rate was increased to the fastest possible. The result was an increase in the limiting shear transfer from 0.23 ksf (11 kPa) at a slip rate of 0.004 inch (0.10 mm)/sec to 0.49 ksf (23 kPa) at the faster rate (the LVDT had reached a mechanical stop, thus displacement data are not available).

The variation in the shear transfer, the pore pressure, and the radial effective pressure during the variable-rate test are shown in Plate 5.32. As seen in the plate, the increased rate of slip resulted in a further decrease in the pore pressure, and thus an increase in the calculated values of radial effective pressure.

During the two reversals of loading, the radial pressures returned to values near those recorded prior to loading. As plastic slip was again enforced, the radial pressures changed rapidly, with a decrease of more than 2.1 ksf (100 kPa) in the pore pressure, and a corresponding increase of 2.4 ksf (115 kPa) in the radial effective pressure.

Upon completion of the variable-rate test, the probe was removed from the boring.

5.3.4 Experiment 4

Experiment 4 at the 148-ft (45.1 m) depth was performed in order to obtain values of shear transfer at times after driving which differed from those performed during 1982. These tests would aid in the development of a relationship describing the increase in shear transfer capacity with time, both during undisturbed consolidation and after complete cyclic degradation during cyclic tests performed at various stages of consolidation.

For this experiment, a probe was installed on 12 December 1983 at 14:50. The probe was subjected to one cycle of large-displacement loading 15 minutes after driving followed by a series of large-displacement two-way cyclic loadings at times of 255, 2659, and 5782 minutes after installation.

The variations in soil pressure during consolidation are given in Plate 5.33. As noted for each of the earlier experiments, the breaks in the curves correspond to the times and durations of each of the series of tests.

As shown in Plate 5.33, the total pressure immediately after driving was 17.30 ksf (828 kPa), with a measured pore pressure of 16.30 ksf (780 kPa). During the initial four hours, the pore pressure continued to increase, being 17.03 ksf (815 kPa) just prior to the first load test. The total pressure at this time was 17.54 ksf (839 kPa), for a calculated value of 0.51 ksf (24 kPa) for the radial effective pressure.

During the period prior to the second series of cyclic load tests (2659 minutes after installation), the pore pressure increased to 16.68 ksf (798 kPa), with a simultaneous increase in the total pressure to 17.60 ksf (842 kPa). As shown in Plate 5.33, the combined changes resulted in an increase in the calculated radial effective pressure, from 0.51 ksf (24 kPa) to 0.92 ksf (44 kPa).

Upon completion of the second series of cyclic tests, the total pressure was 17.29 ksf (827 kPa), with the pore pressure being 16.85 ksf (806 kPa). During the subsequent period of reconsolidation, the total pressure increased to 17.9 ksf (857 kPa), with a slight decrease in the pore pressure, to 16.7 ksf (799 kPa), yielding an increase in the radial effective pressure from 0.44 (21 kPa) to 1.2 ksf (57 kPa).

The reasons for the unusual consolidation behavior observed during this experiment are not known. Had only the pore pressure exhibited such response, it would suggest improper saturation of either the porous stone or the cavity between the porous stone and the pressure transducer. Since both the total pressure and the pore pressure showed simultaneous slow increases during the early stages of consolidation, with the pore pressure finally decreasing during the period following the initial series of cyclic tests, such an explanation is not valid.

Had the shear transfer recorded during the immediate test or the first series of cyclic tests been abnormally lower than was measured during the other experiments, then lateral pile movement, or postholing, during driving, with incomplete pile-soil contact, would be suspected; however, the shear transfer recorded during this experiment was consistently larger than the values recorded during the other experiments, indicating that the probe was in intimate contact with the soil.

The shear transfer recorded during the load test performed 15 minutes after driving was 0.20 ksf (10 kPa), which was reduced to 0.18 ksf (9 kPa) on the second loading in tension, shown in Plate 5.34.

The results of the load tests which were begun 255 minutes after driving are shown in Plate 5.34, which contains the tension-loading portions of the first and the eighth cycles of loading to large reversed displacements. The peak shear transfer on the first cycle was 0.40 ksf (19 kPa), with a reduced post-peak value of 0.38 ksf (18 kPa). After eight cycles, the peak resistance was 0.38 ksf (18 kPa), equal to the post-peak resistance on the first cycle, with a value of 0.37 ksf (18 kPa) recorded during plastic slip at large displacements.

Thus, only minor cyclic degradation was observed. This behavior is quite different from that observed during the first three experiments, where losses of 33 to 37 percent of the static capacity were observed.

The simultaneous variations in the pore pressure, the radial effective pressure, and the shear transfer are given in Plate 5.35. Again, the abrupt changes in the radial pressures accompany plastic slip; the values tend to return to near the static values recorded prior to the test during the periods of reversal in load direction.

Static pressure measurements at the end of the cyclic loading indicated a total pressure of 17.41 ksf (833 kPa), a decrease of 0.13 ksf (6 kPa) from the static total pressure recorded prior to loading; a value of pore pressure of 17.09 ksf

(818 kPa), an increase of 0.06 ksf (3 kPa). The combined changes in the measured pressures yields a decrease of 0.19 ksf (9 kPa) in the radial effective pressure, to 0.32 ksf (15 kPa).

As noted earlier, the pore pressures recorded following this test series began decreasing, being 16.68 ksf (798 kPa) just prior to the next test series. At the same time, the total pressure increased slightly, to 17.60 ksf (842 kPa), yielding an increase in the radial effective pressure from 0.32 (15 kPa) to 0.91 ksf (44 kPa).

Portions of the results of the load tests which were begun 2659 minutes after driving are shown in Plate 5.36, which contains the tension-loading portion of the first cycle of loading. The maximum shear transfer on the first cycle was 0.50 ksf (24 kPa), which reduced to approximately 0.42 ksf (20 kPa) on the tenth cycle (not shown). Again, little effect of cyclic loading on the shear transfer was experienced.

During this load test, the wave action on the platform resulted in severe distortion of the shear-displacement curves, as shown in Plate 5.37, which is the analog record of the sixth cycle of loading. During this load test, the wave heights were approximately 6 (1.8) to 8-ft (2.4 m), and resulted in lateral sway and tilting of the structure. The vertical component of the motion of the structure resulted in uneven loading of the probe. The movement of the probe even reversed during the plastic-slip portion of the test, as can be seen in the small hysteresis loops along the upper and lower portions of the analog record.

The effects of such motion on the results of the tests are not significant, as shown in Plate 5.37. However, the effects were exaggerated in the digital data.

A comparison of the maximum shear transfer recorded on the first cycle of this test with that recorded during the earlier tests shows that the soil had fully recovered from the earlier cyclic degradation, with increases noted in both the initial maximum and the cyclic minimum shear transfer. Again, the behavior was unlike that observed during the first three experiments, with only minor losses in resistance being recorded.

The simultaneous variations in the pore pressure, the radial effective pressure, and the shear transfer are given in Plate 5.38. Again, the apparent raggedness of the curves contained in the plate are due to the motion of the structure. It should again be noted that the effects of such motion do not appreciably affect the measured response or the interpretation of the results.

The effects of the cyclic loading of the probe on the static pressures which were measured included a decrease in the total pressure to 17.29 ksf (827 kPa), and an increase in the pore pressure to 16.85 ksf (806 kPa), resulting in a decrease in the radial effective pressure from 0.91 ksf (44 kPa) to 0.44 ksf (21 kPa).

During a subsequent period of time allowed for reconsolidation, the total pressure fluctuated due to the effects of the motion of the probe, yet exhibited a tendency to increase, being 17.87 ksf (855 kPa) at the end of the reconsolidation period. The pore pressure during this period also fluctuated, with a net decrease to 16.70 ksf (799 kPa) prior to the initiation of the next series of load tests. The combined changes in the total and the pore pressures resulted in an increase in the calculated value of radial effective pressure, to 1.17 ksf (56 kPa).

Portions of the results of the load tests performed 5782 minutes after driving are shown in Plate 5.39, which contains the tension-loading portions of the first and tenth cycles of loading to large reversed displacements. The peak shear transfer on the first cycle was 0.52 ksf (25 kPa), with 0.49 ksf (23 kPa) being recorded on the tenth cycle.

The variations in the pore pressure, the radial effective pressure, and the shear transfer during the series of cyclic tests are shown in Plate 5.40. As seen in the plate, the values of pore pressure and radial effective pressure during the reversals of loading were near the pre-test static values; only during plastic slip did the pressures vary significantly, with decreases in the pore pressure, and simultaneous increases in the radial effective pressure, accompanying plastic shearing of the soil.

Static pressure measurements made at the end of the cyclic tests show the following effects: a decrease in the total pressure to 17.57 ksf (841 kPa), a slight increase in the pore pressure to 16.73 ksf (801 kPa), and a decrease in the radial effective pressure to 0.84 ksf (40 kPa). During a 10-minute period in which the loading system was removed, both the total and the pore pressures increased slightly, to 17.62 ksf (843 kPa) and 16.80 ksf (804 kPa), respectively, indicating very little change in the radial effective pressure.

The probe was then removed from the boring, with no further pressure measurements being made.

The behavior observed during this experiment, with respect to both the pressure response during consolidation and to the shear transfer behavior under cyclic loading, was completely unlike the behavior observed in the three earlier experiments, almost as if the experiments were performed in a completely different soil.

5.3.5 Experiment 5

Experiment 5 at the 148-ft (45.1 m) depth was performed using the 1.72-inch (4.37 cm)-diameter X-probe. This experiment was performed in order to obtain data by which the effects of diameter and of tip plugging could be observed. In this manner, the results of Experiment 2 could be compared with a full-displacement probe; it had earlier been postulated that the 3-inch (7.62 cm)-diameter probes at the 58 (17.7) and 148-ft (45.1 m) depths in the experiments with premature plugging would behave as fully-plugged piles.

For this experiment, the probe was installed by pushing it into place on 13 December 1983 at 12:34. The probe was loaded to failure in tension and compression 17 minutes after installation, then subjected to large-displacement two-way cyclic tests 75 and 1500 minutes after installation. The consolidation times were chosen in order to have approximately the same degree of consolidation as had been used for the 3-inch (7.62 cm)-diameter model, assuming that the consolidation time varied directly with the squares of the diameters of the probes.

The variations in soil pressure during consolidation are shown in Plate 5.41. Again, the times of the load tests are clearly shown in the plate by the discontinuities in the curves. The pressures developed by the full-displacement probe were larger than those developed by the partial-displacement 3-inch (7.62 cm) diameter probes, with a maximum total pressure of 24.13 (1155 kPa) ksf and a maximum pore pressure of 22.62 ksf (1082 kPa) being recorded immediately after installation.

The pore pressures dissipated rapidly, being 21.52 ksf (1030 kPa) at the beginning of the first load test, with a value of 23.57 ksf (1128 kPa) for the total pressure. After 75 minutes, the total radial pressure was 22.92 ksf (1097 kPa) and the pore pressure was 20.27 ksf (970 kPa), yielding a value of 2.65 ksf (127 kPa) for the radial effective pressure.

Following the cyclic load test performed 75 minutes after installation, the soil was allowed to consolidate until 1500 minutes had elapsed, at which time the total radial pressure was 21.15 ksf (1012 kPa) and the pore pressure was 16.84 ksf (806 kPa), indicating a value of 4.32 ksf (207 kPa) for the radial effective pressure.

During the load test performed 17 minutes after installation, the value recorded for the shear transfer was 0.28 ksf (13 kPa). During this load test, the soil anchor at the tip of the probe slipped, with no relative displacement between the probe and the tip being recorded. Thus, no shear transfer curves are available for this load test.

The results of the load tests performed 75 minutes after installation are shown in Plate 5.42, which contains the tension-loading portion of the first and fifth cycles of loading to large reversed displacements. The peak shear transfer on the first cycle was 0.52 ksf (25 kPa), with a reduced post-peak value of 0.40 ksf (19 kPa). No losses in resistance were observed during these tests, with a peak value of shear transfer of 0.52 ksf (25 kPa) on the fifth cycle, and with a reduced post-peak value of 0.41 ksf (20 kPa). Again, the behavior observed during this experiment was completely unlike that observed in the first three experiments. It was, however, very similar to that observed in Experiment 4.

It is interesting to note, however, that there was an increase in the yield displacement, from 0.007 inch (0.18 mm) on the first loading to 0.014 inch (0.35 mm) on the fifth cycle.

The variations in the pore pressure, the radial effective pressure, and the shear transfer during these cyclic tests are shown in Plate 5.43. Again, the pattern of response was similar to that observed earlier for the 3-inch (7.62 cm)-diameter probes, with the pore pressure decreasing (and the effective pressure increasing) during plastic slip with constant shear transfer.

At the end of the load test, the total pressure was 22.54 ksf (1079 kPa), a decrease of 0.38 ksf (18 kPa). The pore pressure was 20.31 ksf (972 kPa), an increase of 0.04 ksf (2 kPa), yielding a decrease in the radial effective pressure of 0.42 ksf (20 kPa).

The results of the load tests performed 1500 minutes after installation are shown in Plate 5.44, which contains the tension-loading portion the first and ninth cycles at large displacements. During these load tests, the motions of the structure due to wave loadings again affected the tests, with the rate of loading being quite variable during the cycling.

The maximum shear transfer on the first cycle was 0.77 ksf (37 kPa), with no post-peak reduction in resistance. Thus, the soil had continued to increase in shear transfer capacity during the period of consolidation. Although no cyclic degradation in the peak resistance was observed, reductions were noted in the post-peak resistance.

A comparison of the two curves in Plate 5.44 shows an interesting pattern of behavior. During the first loading, the soil exhibits elastic-plastic response at failure. After load reversal, the soil exhibits a peak-residual response, with a reduction in post-peak resistance from 0.76 ksf (36 kPa) to 0.59 ksf (28 kPa), a reduction of 22 percent.

The variations in the pore pressure, the radial effective pressure, and the shear transfer during the cyclic tests are given in Plate 5.45. As seen in the plate, the pore pressure showed a general increase during the tests, with an accompanying decrease in the radial effective pressure.

Upon the conclusion of these tests, the probe was removed from the boring, with no static pressure measurements being made.

5.3.6 Summary of Results at the 148-ft (45.1 m) Depth

The results of the load tests performed during the experiments at the 148-ft (45.1 m) depth are given in the following table. Again, the reported soil pressures are the static pressures measured before and after the active load tests, and represent actual pressures in the soil adjacent to the pile.

5.2 RESULTS OF THE EXPERIMENTS AT THE 148-FT (45.1 M) DEPTH

Experiment Number	Date	Time	Elapsed Time (min)	Soil Pressures Total ksf	Pore (kPa)	Shear Transfer / Cycle Num ksf (kPa)
1	04 Dec 1982	17:29	11	18.48	18.49	0.28 / 1
				(884)	(885)	(13)
	07 Dec 1982	13:19	4081	18.21	17.09	0.36 / 1
				(871)	(818)	(17)
		14:10	4132	17.98	17.38	0.24 / 10
				(860)	(832)	(11)
2	02 Dec 1982	14:18	8	20.41	19.55	0.24 / 1
				(977)	(935)	(11)
		22:44	514	19.82	18.15	0.41 / 1
				(948)	(885)	(20)
	03 Dec 1982	00:21	611	19.65	18.29	0.30 / 12
				(940)	(875)	(14)
		07:03	1013	19.30	17.34	0.42 / 1
				(924)	(830)	(20)
3	06 Dec 1982	11:52	6	18.46	18.17	0.24 / 1
				(883)	(869)	(11)
		20:07	501	18.40	17.54	0.35 / 1
				(880)	(839)	(17)
		20:54	548	18.33	17.28	0.22 / 12
				(877)	(827)	(11)
	07 Dec 1982	08:14	1228	18.13	17.21	0.39 / 1
				(868)	(823)	(19)
		08:52	1266	18.20	17.37	0.25 / 10
				(871)	(831)	(12)

	08 Dec 1982	07:44	2638	18.17	17.00	0.30 / 1
				(869)	(813)	(14)
		08:12	2666	18.31	16.90	0.26 / 6
				(876)	(809)	(12)
4	12 Dec 1983	15:05	15	17.36	16.52	0.20 / 1
				(831)	(790)	(10)
		19:05	255	17.54	17.03	0.40 / 1
				(839)	(815)	(19)
		20:18	328	17.41	17.09	0.38 / 8
				(833)	(818)	(18)
	14 Dec 1983	11:09	2659	17.60	16.68	0.50 / 1
				(842)	(798)	(24)
		12:21	2731	17.29	16.85	0.42 / 10
				(827)	(806)	(20)
	16 Dec 1983	15:12	5782	17.87	16.70	0.52 / 1
				(855)	(799)	(0.25)
		16:22	5852	17.57	16.73	0.49 / 10
				(841)	(801)	(23)
5	13 Dec 1983	12:51	17	23.24	21.35	0.28
				(1112)	(1022)	(13)
		13:49	75	22.92	20.27	0.52 / 1
				(1097)	(970)	(25)
		14:41	127	22.54	20.31	0.52 / 5
				(1079)	(972)	(25)
	14 Dec 1983	13:34	1500	21.15	16.84	0.77 / 1
				(1012)	(806)	(37)
		14:15	1541	20.75	17.97	0.76 / 9
				(993)	(860)	(36)

A comparison of the values of peak shear transfer recorded after the initial period of undisturbed consolidation during Experiments 1, 3, and 4 agreed reasonably well. For periods of consolidation ranging from 250 to 4080 minutes, the values of peak shear transfer varied from 0.35 (17 kPa) to 0.40 ksf (19 kPa), a variation of only 13 percent.

For the same periods of consolidation, the values of radial effective pressure varied from 0.51 ksf (24 kPa) to 1.13 ksf (54 kPa). It is of interest to note that the largest value of shear transfer, 0.40 ksf (19 kPa), was observed for the probe with the smallest value of radial effective pressure, 0.51 ksf (24 kPa). Values of peak shear transfer of 0.35 (17) and 0.36 ksf (17 kPa) were recorded during the experiments in which the radial effective pressures prior to the load tests were 0.87 (42) and 1.13 ksf (54 kPa).

The values of peak shear transfer measured during Experiment 2 (in which the cutting shoe was allowed to plug prematurely) also agreed reasonably well with the values measured in Experiments 1, 3, and 4, in which the cutting shoe did not plug prematurely even though the magnitude of the radial effective pressure, 1.67 and 1.96 ksf (80 and 94 kPa) were approximately double those in Experiments 1, 3, and 4 at comparable times after driving.

Based solely on the agreement shown during these experiments (1 through 4) it may be concluded that, in the experiments at the 58-ft (17.7 m) depth, the values of shear transfer which were observed are not significantly different from those which would have been recorded had the proper installation procedures been followed. However, the values of radial effective pressure, and the consolidation times, may have been different.

During Experiment 5, values of shear transfer which were measured using the X-probe were larger than those measured using the partial-displacement 3-inch (7.62 cm)-diameter probes, with 0.52 ksf (25 kPa) being recorded after 75 minutes of consolidation, and 0.77 ksf (37 kPa) after 1500 minutes. It should be noted that the corresponding values of radial effective pressure were also significantly larger than was observed for the 3-inch (7.62 cm)-diameter tools, being 2.65 ksf (127 kPa) and 4.32 ksf (207 kPa) prior to the load tests.

During the first three experiments, losses of approximately one-third of the static capacity were recorded during the cyclic tests. Although the losses in resistance were found to be temporary in nature, the losses in resistance for a foundation pile would coincide with the design wave loads which the pile would be required to sustain. In the design of axial piles for cyclic loading, such losses in resistance in the soil along the pile must be considered, and estimates of reduced pile capacities used in estimating the actual factors of safety under design load conditions.

The degree of recovery of shear transfer capacity following periods of reconsolidation suggests that, if cyclic degradation occurs in the early stages of consolidation, the phenomenon may be only temporary in nature. In each instance, the shear transfer capacity was at least partially recovered. Had a longer period of time been allowed for recovery during Experiment 3, the soil may have also recovered completely.

An examination of the soil pressure data indicates that the often purported linear relationships between the magnitudes of radial effective pressure and the static or cyclic minimum shear transfer capacity of the soil were not observed.

Based on consideration of the patterns of pore pressure changes during cyclic loading, coupled with the consideration of the effects of changes in moisture content and void ratio on shear strength, concepts for the mechanism of cyclic degradation in axial capacity, and for the recovery in capacity during periods of reconsolidation, were developed, at least in a qualitative sense.

In order to compare the soil pressures which were measured in the experiments, it is necessary to examine the conditions under which the experiments were performed.

During Experiment 2, the cutting shoe was intentionally allowed to plug when the bottom of the shoe was at the mid-depth of the zone of soil which was tested. As expected, the probe developed higher lateral pressures than were observed in Experiments 1, 3, and 4, which had unplugged thin-walled cutting shoes.

Experiment 5 was performed with the 1.72-inch (4.37 cm)-diameter X-probe, and showed even higher values of excess radial pressures generated during installation than had the 3-inch (7.62 cm) probe in Experiment 2.

A comparison of the values of radial effective pressures which were observed at high degrees of consolidation during the experiments shows that, indeed, the values of pressure observed during Experiment 2 were less than would be expected for a fully-plugged pile; the pressures were, however, larger than those developed using a thin-walled cutting shoe. During Experiments 1, 3, and 4, the radial effective pressures at high degrees of consolidation were 1.12 (54), 1.17 (56), and 1.17 ksf (56 kPa), respectively. During Experiment 2, the radial effective pressure after 1010 minutes of consolidation was 1.96 ksf (94 kPa); the effective pressure would have continued to increase with additional consolidation, but would likely not have reached a value as large as the value of 4.32 ksf (207 kPa) which was observed for the full-displacement X-probe.

Based on the behavior observed in the five experiments at this depth, it may be concluded that the final value of radial effective pressure which will be developed in the soil adjacent to a pile is a function of the volume of soil which was displaced, suggesting that the radial pressures which develop are some function of the wall thickness of the pile.

5.4 The Experiments at the 178-ft (54.3 m) Depth

Four experiments were performed at the 178-ft (54.3 m) depth, three with the 3-inch (7.62 cm)-diameter pile segment models with thin-walled cutting shoes, and one with the 1.72-inch (4.37 cm)-diameter pile segment model, the X-probe. All the experiments at this depth were performed in addition to those originally scheduled under the CNRD 13-2 and 13-3 contracts.

One experiment was added to the nine which were performed at depths of 58 (17.7), 148 (45.1), and 208-ft (63.4 m) below the seafloor in 1982, under CNRD Research Project 13-2. Three more experiments at this depth were added to the three long-term experiments scheduled in 1983 under CNRD Research Project 13-3.

The experiment in 1982 (CNRD 13-2) was added at the request of one of the observers, so that the behavior of a probe under load-controlled cycling could be compared to the response under displacement-controlled cyclic loading, which was the primary form of pre-failure cyclic testing, used in order to provide more precise control of the tests.

It should be noted that, in modelling the behavior of a short segment of a long flexible pile, controlled-displacement loading more closely duplicates the conditions which exist at the pile-soil interface, since the relative displacement of any section of the pile is primarily determined by the elastic stiffness of the length of pile embedded below that section.

Load-controlled cycling more closely duplicates the response at the top of a short, relatively stiff pile, such as would be used for onshore foundations.

The interpretation of the results of controlled-displacement cyclic load tests, and the extension of those results to the behavior of segments of long, elastic piles is thus much more straightforward than is the extension of the results of load-controlled cyclic tests.

The experiments performed in 1983 were added to the test program in order to obtain more data from the important Stratum III soil, which lay below a depth of 160-ft (48.8 m) below the mudline. Since the dominant portion of the capacity of the 30-inch (76.2 cm)-diameter test pile would be developed in this soil, it was felt that the additional experiments would prove to be valuable in the later interpretation of the results of the large-diameter pile tests.

The experiments included one which consisted of large-displacement cyclic tests at similar degrees of consolidation and one at a later stage of undisturbed consolidation than the experiment performed in 1982; the experiments were performed with 3-inch (7.62 cm)-diameter probes having thin-walled cutting shoes.

In addition, one experiment was performed with large-displacement cyclic loading at a high degree of consolidation using the 1.72-inch (4.37 cm)-diameter probe, in order to observe the effects of diameter and plugging on consolidation and the time-rate of increase in shear transfer.

5.4.1 Experiment 1

Experiment 1 was performed at the request of a representative of the Norwegian Geotechnical Institute, who desired that an experiment be performed with load-controlled cyclic testing, rather than with controlled displacements. The experiment was performed in order to determine whether the onset of cyclic degradation in resistance was similar for the different methods used for test control. In addition, a large-displacement cyclic test was performed after a period of reconsolidation, in order to observe the effects of time and reconsolidation on the recovery of shear transfer capacity.

For this experiment, a 3-inch (7.62 cm)-diameter probe was installed by driving, with the installation being completed at 11:44 on 8 December 1982. The probe was subjected to a large-displacement test in tension with one cycle of load reversal 7 minutes after driving. Consolidation was then allowed to proceed for 504 minutes, at which time a series of load-controlled tests were begun. Repeated tension loads were applied, with the difference between the maximum and minimum loads being increased until failure was achieved by progressive pullout of the probe. The probe was then subjected to ten cycles of reversed large-displacement loading to accomplish full cyclic degradation in resistance. Consolidation was then allowed to continue until 1173 minutes had elapsed since installation, when the probe was subjected to twelve cycles of reversed large-displacement loading, then removed from the borehole.

The variations in the soil pressures during consolidation are shown in Plate 5.46. The breaks, or discontinuities, in the curves correspond to the times at which the load tests were performed. As shown in the Plate, the maximum total pressure recorded after driving was 24.38 ksf (1167 kPa), with a maximum pore pressure of 23.55 ksf (1127 kPa). Prior to the load test performed 504 minutes after driving, the total pressure had decreased to 23.44 ksf (1122 kPa) and the pore pressure had decreased to 20.81 ksf (996 kPa), for a calculated value of 2.63 ksf (126 kPa) for the radial effective pressure. Following the cyclic load test, the total and pore pressures continued to decrease, with a measured value of 22.99 ksf (1100 kPa) for the total pressure and 20.50 ksf (981 kPa) for the pore pressure 1173 minutes after driving, yielding a value of 2.49 ksf (119 kPa) for the radial effective pressure immediately prior to the final cyclic load tests.

During the load test performed 7 minutes after driving, a maximum value of 0.36 ksf (17 kPa) was recorded for the shear transfer (not shown), which reduced to 0.20 ksf (10 kPa) after the load reversal. The tensile-loading portion of the second load reversal of loading is contained in Plate 5.47.

The results of the load-controlled cyclic tests are given in Plate 5.48, which shows the cycle-by-cycle variations in the peak values of shear transfer, along with the corresponding values of displacement. As seen in the Plate, the probe failed by abrupt pullout at a value of shear transfer of 0.70 ksf (33 kPa). Little evidence of cyclic degradation can be seen in the figure; had losses in capacity been occurring, the values of displacement associated with a given set of load levels would have diverged. Although progressive upward movement can be seen for the two highest load levels (beginning on the hundredth cycle of loading), the peak-to-peak range of displacement is essentially constant.

The results of portions of the cyclic tests are shown in Plate 5.47. The upper curve shows the final cycle of the one-way cyclic test, in which the peak shear transfer was 0.70 ksf (33 kPa). The tension-loading portions of the first and tenth cycles of the large-displacement tests are also shown in the plate. As shown in the Plate, the maximum shear transfer on the first cycle of large-displacement loading was 0.56 ksf (27 kPa), somewhat less than that recorded on the first pullout. After ten cycles of reversal, the limiting shear transfer had been reduced to 0.40 ksf (19 kPa), a loss of 30 percent of the shear transfer recorded on the first cycle.

Not shown in the plate is an intermediate loading to failure in tension, following the one-way cyclic test, in which the peak shear transfer was 0.60 ksf (29 kPa).

The variations in the pore pressure, the radial effective pressure, and the shear transfer during the complete series of cyclic tests are shown in Plate 5.49. As shown in the plate, the load-controlled cycling resulted in a decrease in the radial effective pressure, which was not due solely to increases in the pore pressure. The first abrupt decrease in pore pressure accompanied the initial plastic slip in the soil; the later fluctuations accompanied the reversed large-displacement cyclic loading.

At the end of the load test, the pore pressures increased for the first three minutes, from a value of 21.00 ksf (1005 kPa) at the end of the test to a peak value of 21.36 ksf (1022 kPa), with the pressures then decreasing. Again, this behavior suggests an initial flow of pore water toward the shear zone, until the depression in the pore pressure gradient had been alleviated; whereupon the radial flow of pore water reversed, with the pore water again moving away from the pile.

The excess pore pressures again quickly dissipated (although less rapidly than had the depressed pressures), reaching the value measured prior to the test in 48 minutes (45 minutes after reaching the peak value). As seen in Plate 5.46, the consolidation path then returned to that established prior to the disturbance.

Again, the pattern of response agrees well with the concepts developed for the behavior observed at the 148-ft (45.1 m) depth. The pattern of pore pressure variations after the end of loading again indicate the superposition of two separate pore pressure effects, one for the near-pile clay which is enclosed within the surface of slip, and one for the clay within the shear zone, in which large plastic deformations occur.

During a period of 478 minutes of reconsolidation following the cyclic tests (for a total of 1173 minutes after installation), the total pressure remained reasonably constant, near 23.0 ksf (1101 kPa), with an accompanying decrease in the pore pressure to 20.50 ksf (981 kPa), yielding an increase in the radial effective pressure to 2.50 ksf (120 kPa).

The results of the large-displacement cyclic tests performed after 1173 minutes of consolidation are shown in Plate 5.50. Again, only the tension-loading portion of the curves are shown, for the first and twelfth cycles. During these tests, the maximum shear transfer on the first cycle was 0.72 ksf (34 kPa), with a minimum value of 0.46 ksf (22 kPa) on the twelfth cycle.

Thus, the increase in the peak shear transfer, from 0.40 (19 kPa) to 0.72 ksf (34 kPa), must be associated with the dissipation of the excess pore pressures due to the volume changes in the clay within the shear zone, with the clay returning to almost the same condition that existed prior to the cyclic degradation in the first series of tests.

The variations in pore pressure, radial effective pressure, and shear transfer during the cyclic tests are shown in Plate 5.51. As seen in the plate, significant decreases in the pore pressure (and increases in the radial effective pressure) accompanied plastic slip in compression, with only small decreases (and increases) being observed during plastic slip in tension. In general, however, the pressures fluctuated above and below the static pressures recorded prior to the test.

Within minutes after the end of the cyclic tests, the probe was subjected to a large-displacement test in tension, with the load rate increased during plastic slip. As shown in Plate 5.52, the shear transfer increased from 0.40 ksf (19 kPa) at a slip rate of 0.001 inch (0.025 mm)/sec to 0.47 ksf (22 kPa) at a slip rate of 0.012 inch (0.30 mm)/sec, an increase of approximately 18 percent with a ten-fold increase in the rate of slip. The rate was then decreased to 0.001 inch (0.025 mm)/sec at a displacement of approximately 0.2 inch (5 mm), whereupon the shear transfer reduced to near the earlier value at the slow rate.

The simultaneous variations in the pore pressure, radial effective pressure, and the shear transfer during the variable-rate test are shown in Plate 5.53. As shown in the plate, significant fluctuations in pressure were observed, with decreases in the pore pressure from 20.71 ksf (991 kPa) prior to yield to a minimum of 18.78

ksf (899 kPa) during the fast-rate plastic slip. At the same time, the radial effective pressure increased from a pre-yield value of 2.43 ksf (116 kPa) to a maximum value of 4.86 ksf (233 kPa), again during the fast-rate plastic slip.

At the conclusion of the variable-rate test, the hydraulic ram was extended to its full stroke to break the cutting shoe free, and the probe was removed from the boring without any observations of the static soil pressures after the test.

5.4.2 Experiment 2

Experiment 2 at the 178-ft (54.3 m) depth was to be used to obtain values of peak shear transfer at similar degrees of consolidation as had been allowed for Experiment 1, in order to ascertain the effects of load-controlled cycling on the initiation of cyclic degradation. In addition, it had been intended to obtain additional data for an investigation of the reconsolidation and recovery in shear transfer after cyclic degradation early in the consolidation history. Unfortunately, the seals around one of the pressure transducers failed, so that additional load tests were not performed.

For this experiment, a 3-inch (7.62 cm)-diameter probe was installed on 17 December 1983, at 12:28. The probe was tested to failure in tension and compression 8 minutes after driving, with a reversed large-displacement cyclic test performed 496 minutes after driving.

As noted earlier, leakage of moisture into the probe resulted in unstable readings from the pressure transducers. The shear transfer and displacement measurements were not affected. However, the pressure measurements made during the latter part of consolidation and during the load test are not usable.

The variations in soil pressure during consolidation are shown in Plate 5.54. Again, the breaks in the curves correspond to the time of the load test performed within minutes after driving. As seen in the Plate, the total pressure transducer became unstable approximately 80 minutes after driving. The data taken prior to the instability showed a maximum total pressure after driving of 22.76 ksf (1089 kPa), with a maximum value of pore pressure of 22.18 ksf (1061 kPa). At the time the measurements became unreliable, the total pressure had decreased to 21.13 ksf (1011 kPa) and the pore pressure had decreased to 20.35 ksf (974 kPa), for a calculated value of 0.78 ksf (37 kPa) for the radial effective pressure.

During the load test performed 8 minutes after driving, shown in Plate 5.55, the shear transfer was 0.10 ksf (5 kPa) on the second loading to failure in tension after the reversal of loading.

The results of the large-displacement, two-way cyclic tests are also shown in Plate 5.55, which contains the tension-loading portion of the first, the fifth, and the tenth cycles of loading. The maximum shear transfer on the first cycle was 0.53 ksf (25 kPa), which was reduced to 0.44 ksf (21 kPa) on the tenth cycle.

Since the pressure transducers were no longer functional on the probe, it was removed shortly after the load test.

5.4.3 Experiment 3

Experiment 3 at the 178-ft (54.3 m) depth was performed with the objective of observing the long-term consolidation of the soil around a thin-walled pile model. The load tests were performed in order to obtain a value of shear transfer after long-term undisturbed consolidation. The results of this experiment can then be compared with those from Experiments 1 and 2 to observe the increase in shear transfer with time.

For this experiment, a 3-inch (7.62 cm)-diameter probe was installed on 13 December 1983, at 17:15. The probe was subjected to one reversal of large-displacement loading 8 minutes after installation and a large-displacement, two-way cyclic test 4290 minutes after installation, and was then removed from the boring.

The variations in soil pressure during consolidation are shown in Plate 5.56. The discontinuities in the curves correspond to the time and duration of the load test performed 8 minutes after driving. The maximum total pressure immediately after driving was 25.29 ksf (1210 kPa), with a maximum pore pressure of 24.94 ksf (1193 kPa). Both the total and pore pressure decreased with time, with the total pressure being 22.93 ksf (1097 kPa) and the pore pressure being 19.86 ksf (950 kPa) after 4290 minutes of consolidation, yielding an increase in the radial effective pressure to 3.07 ksf (147 kPa) at the time the probe was load-tested.

During the load test performed 8 minutes after driving (not shown), the maximum shear transfer was 0.22 ksf (11 kPa), with a value of 0.09 ksf (4 kPa) being recorded after the reversal of loading.

The results of the large-displacement, two-way cyclic tests which were begun 4290 minutes after driving are shown in Plate 5.57. The Plate contains the tension-loading portions of the first, seventh, and tenth cycles. The maximum shear transfer on the first cycle was 0.66 ksf (32 kPa); the maximum shear transfer on the tenth cycle was 0.44 ksf (21 kPa). As seen in the Plate, the process of cyclic degradation in resistance was essentially complete after less than ten cycles. The shear transfer on the second cycle of loading was 0.47 ksf (23 kPa), suggesting that the greatest damage to the soil occurred after only one reversal of loading.

The results of this experiment, when compared with those from Experiments 1 and 2, indicate that the shear transfer increased rapidly after driving. Values of 0.53 (25) and 0.70 ksf (34 kPa) were recorded after approximately four hours of consolidation in the first two experiments, as compared with 0.66 ksf (32 kPa) in this experiment after 72 hours of consolidation.

The simultaneous variation in the pore pressure, the radial effective pressure, and the shear transfer are shown in Plate 5.58. Disturbance created by attaching the hydraulic ram to the string of N-rods altered the pressures in the soil, with the total pressure decreasing from 22.93 ksf (1097 kPa) to 22.88 ksf (1095 kPa), and the pore pressure increasing from 19.86 ksf (950 kPa) to 20.40 ksf (976 kPa), resulting in a decrease in the calculated value of radial effective pressure from 3.08 (147) to 2.48 ksf (119 kPa). During the cyclic loading, the decreases in the pore pressure and the increases in radial effective pressure correspond to periods of plastic slip.

The cyclic load test had a significant effect on the static radial pressures. Readings made only seconds after the cessation of loading showed a value of 22.62 ksf (1082 kPa) for the total pressure, a reduction of 0.31 ksf (15 kPa) below the pre-test static pressure, a value of 21.39 ksf (1024 kPa) for the pore pressure, an increase of 1.53 ksf (73 kPa) above the pre-test static pressure. The combined changes in the measured pressures yields a reduction in the calculated radial effective pressure to 1.24 ksf (59 kPa), a decrease of 1.83 ksf (88 kPa).

Following the test, the excess pore pressures rapidly dissipated. After only 15 minutes, the value of total pressure was 22.58 ksf (1080 kPa) and the pore pressure was 20.55 ksf (983 kPa), indicating an increase in the radial effective pressure to 2.04 ksf (98 kPa). The rapid rate of dissipation again indicates that the excess pore pressures which arise from the cyclic loading must extend only a short radial distance from the pile wall.

At the end of the 15-minute period, during which the loading system was being disconnected, the probe was removed from the boring.

5.4.4 Experiment 4

For Experiment 4 at the 178-ft (54.3 m) depth, a 1.72-inch (4.37 cm)-diameter (X-probe) pile segment model was installed on 14 December 1983, at 17:52. The probe was immediately subjected to a reversed large-displacement test, starting as the probe was being pushed into place using the hydraulic loading system. Following this test, consolidation was allowed to proceed for 1442 minutes, at which time the probe was subjected to 14 cycles of large-displacement, two-way cyclic loading. Consolidation was then allowed to continue until 2927 minutes had elapsed since installation. The probe was then subjected to 10 cycles of large-displacement, two-way cyclic loading.

The variation in soil pressures during consolidation are shown in Plate 5.59. The maximum total pressure after installation was 28.04 ksf (1342 kPa), with a maximum pore pressure of 25.44 ksf (1217 kPa). As seen in Plate 5.59, both the total and pore pressures decreased with time, being 25.09 ksf (1201 kPa) and 20.37 ksf (975 kPa), respectively, 1442 minutes after installation, yielding a value of 4.72 ksf (226 kPa) for the radial effective pressure. Upon completion of the cyclic load test, the soil was allowed to continue to consolidate until 2927 minutes had elapsed, at which time the total pressure was 24.39 ksf (1167 kPa) and the pore pressure was 19.67 ksf (941 kPa), yielding a radial effective pressure of 4.72 ksf (226 kPa).

During the load test immediately after installation (not shown) the shear transfer was 0.27 ksf (13 kPa).

The results of the load tests begun after 1442 minutes of consolidation are shown in Plate 5.60. The Plate contains the tension-loading portions of the first, second, and tenth cycles of loading. During this experiment, the X-probe exhibited a behavior pattern which differed from that which was observed for the 3-inch (7.62 cm)-diameter probes. The maximum shear transfer on the first cycle was 0.62 ksf (30 kPa), with only a small post-peak reduction in resistance, to 0.59 ksf (28 kPa). Upon the first reversal of loading, the maximum negative shear transfer was -0.78 ksf (-37 kPa), with a peak value in tension on the second cycle of 0.77 ksf (37 kPa). On the tenth cycle, a peak shear transfer at yield of 0.68 ksf (33 kPa) was recorded, with a reduced post-peak value of 0.52 ksf (25 kPa).

Thus, only minor cyclic degradation was noted during this series of cyclic tests.

The variations in the pore pressure, the radial effective pressure, and the shear transfer are given in Plate 5.61. As shown in the plate, the pore pressure increased and the radial effective pressure decreased during the initial loading. The latter remained depressed throughout the loading, although temporary fluctuations occurred during the plastic slip, as had been observed for the 3-inch (7.62 cm)-diameter probes.

Measurements of the static pressures made following the load test indicated a value of 24.62 ksf (1178 kPa) for the total pressure (a decrease of 0.47 ksf (22 kPa)), and a value of 21.40 ksf (1024 kPa) for the pore pressure (an increase of 1.03 ksf (49 kPa)). The resulting radial effective pressure was 3.22 ksf (154 kPa), or a decrease of 1.50 ksf (72 kPa). Following the cyclic load test, the pore pressures rapidly dissipated, with the total pressure exhibiting a much slower rate of decline. The pore pressure had decreased to the pre-test value after a period of 3 hours; due to the combined decreases in both the total and the pore pressures, the radial effective pressure did not reach the pre-test value until 14 1/2 hours after the end of the test.

The shear transfer-displacement behavior recorded during the large-displacement cyclic tests after 2927 minutes of consolidation are shown in Plate 5.62. On the first cycle, the peak shear transfer was 0.81 ksf (39 kPa), with a reduced post-peak value of 0.73 ksf (35 kPa). On the second cycle, the peak shear transfer was 1.00 ksf (48 kPa), with a reduced post-peak value of 0.69 ksf (33 kPa). On the tenth cycle, the peak shear transfer was 0.88 ksf (42 kPa), with a post-peak value of 0.61 ksf (29 kPa).

Again, the effects of load reversal on the shear transfer response can be seen in a comparison of the first cycle of loading to those performed after reversal. The peak shear transfer is considerably greater, with the primary effect of cyclic degradation being apparent only in the value of residual shear transfer recorded after yield.

The shear transfer-displacement behavior during these load tests was also affected somewhat by the results of wave action on the structure. The variation in the peak resistance is partly due to the failure occurring at different rates, especially in the later cycles. The post-peak, or residual, values showed much better repeatability, with little variation after the fifth cycle.

The simultaneous variations in the pore pressure, the radial effective pressure, and the shear transfer during the cyclic tests are shown in Plate 5.63. As noted during each experiment, the simultaneous decreases in pore pressure and increases in the radial effective pressure accompany the plastic shearing of the soil. In general, the radial effective pressure tended to decrease during the experiment, with a parallel increase in the pore pressures, although the effects on the static pressures are masked by the fluctuations during active shear.

Static pressure measurements made at the end of the cyclic load test indicated a value of total pressure of 24.04 ksf (1150 kPa), a reduction of 0.35 ksf (17 kPa), and a value of pore pressure of 20.48 ksf (980 kPa), an increase of 0.81 ksf (39 kPa). The combined reduction in total pressure and increase in pore pressure resulted in a decrease in the radial effective pressure from 4.72 (226) to 3.56 ksf (170 kPa).

The probe then remained in place for 30 minutes, while the loading system was being removed. During this period, the excess pore pressures rapidly dissipated, being 19.96 ksf (955 kPa) at the end of 30 minutes. The total pressure remained fairly constant, being 23.97 ksf (1147 kPa) at the end of the 30-minute period, resulting in an increase in the radial effective pressure to 4.01 ksf (192 kPa).

At the end of the 30-minute period, the probe was removed from the boring.

5.4.5 Summary of Results at the 178-ft (54.3 m) Depth

A summary of the results of the load tests performed during the experiments at the 178-ft (54.3 m) depth is given in the following table. As noted earlier, the soil pressures which are tabulated are those measured before and after the conclusion of active load tests. Since radial pore pressure gradients exist in the soil during active loading, and since the pressures measured at the pile wall are not in equilibrium with those on the failure surface, the values of pressure measured during active shear do not truly represent the actual soil pressures acting along the shear surface. The effects of cyclic loading on the soil pressures can be estimated from the static values given in the table.

TABLE 5.3 RESULTS OF THE EXPERIMENTS AT THE 178-FT (54.3 M) DEPTH

Experiment Number	Date	Time	Elapsed Time (min)	Soil Pressures Total ksf	Pore (kPa)	Shear Transfer / Cycle Num ksf (kPa)
1	08 Dec 1982	11:51	7	24.22 (1159)	23.35 (1117)	0.36 / 1 (17)
		20:08	504	23.44 (1123)	20.81 (996)	0.70 / 194 (33)
		23:23	699	23.26 (1113)	21.00 (1005)	0.40 / 205 (19)
	09 Dec 1982	07:18	1173	22.99 (1100)	20.50 (981)	0.72 / 1 (34)
		07:59	1214	22.86 (1094)	20.50 (981)	0.46 / 12 (22)
2	17 Dec 1983	12:36	8	22.56 (1079)	22.06 (1056)	0.13 / 1 (6)
		20:44	496	N/A	N/A	0.53 / 1 (25)
		21:53	564	N/A	N/A	0.44 / 10 (21)
3	13 Dec 1983	17:23	8	24.72 (1183)	24.69 (1181)	0.22 / 1 (11)

	16 Dec 1983	16:45	4290	22.93 (1097)	19.86 (950)	0.66 / 1 (32)
		17:56	4361	22.62 (1082)	21.39 (1024)	0.44 / 10 (21)
4	14 Dec 1983	17:52	0	28.04 (1342)	25.44 (1217)	0.27 (13)
	15 Dec 1983	17:54	1442	25.09 (1201)	20.37 (975)	0.62 / 1 (30)
		18:47	1495	24.62 (1178)	21.40 (1024)	0.68 / 10 (33)
	16 Dec 1983	18:39	2927	24.39 (1167)	19.67 (941)	0.81 / 1 (39)
		19:16	2964	24.04 (1150)	20.48 (980)	0.88 / 10 (42)

A comparison of the results of the experiments performed with the 3-inch (7.62 cm)-diameter probes with open-end cutting shoes (Experiments 1, 2, and 3) shows that the values of maximum shear transfer which were measured agree reasonably well, with better agreement shown among the values of cyclic minimum shear transfer. It is also interesting to note that the time-rate of increase in shear transfer was very rapid, with the major portion of the increase occurring early in the consolidation process.

The behavior observed in Experiment 1 suggests that, if cyclic degradation occurs in the early stages of consolidation, the losses in resistance are only temporary. Again, no experiments were performed in which cyclic degradation was enforced during the later stages of consolidation, with a period of rest and reconsolidation allowed for recovery and a subsequent load test. Thus, no general conclusions may be reached regarding the ability of the soil to recover shearing resistance under such circumstances.

A comparison of the values of excess total and pore pressure measured at the end of driving indicates reasonable agreement among the experiments performed with the 3-inch (7.62 cm)-diameter probes installed with open-end cutting shoes (Experiments 1, 2, and 3), with considerably higher pressures being developed by the full-displacement X-probe.

The soil surrounding the X-probe also consolidated at a faster rate, partly due to the difference in the diameters of the models. Since no long-term experiments were performed at this depth, the process of consolidation had not proceeded to completion; thus no comparison with the in situ ambient pore pressures or radial total pressures can be made, except that the radial total pressure in the soil surrounding the full-displacement model would have remained larger than the comparable pressures acting on the models with the thin-wall cutting shoes.

5.5 The Experiments at the 208-ft (63.4 m) Depth

Six experiments were performed at a depth of 208-ft (63.4 m) below the mudline; three in 1982 under CNRD Research Project 13-2 and three in 1983-84 under CNRD Research Project 13-3. All of the experiments were performed with the 3-inch (7.62 cm)-diameter probes with thin-walled cutting shoes.

The experiments included large-displacement cyclic tests after 8 hours 30 minutes, 69 hours 18 minutes, and 2467 hours 9 minutes of undisturbed consolidation, progressively-increased small displacement load tests after 7 hours 57 minutes of undisturbed consolidation, and the performance of large-displacement cyclic tests after periods of reconsolidation following cyclic degradation which ranged in length from 11 to more than 2400 hours.

Thus, more experience was obtained at this depth than at any other, with retests performed after cyclic degradation had been enforced in both early and late stages of consolidation, and with tests performed sufficiently long after installation to ensure that the process of consolidation was complete. Since the soil at this depth is a part of the same deposit as the soil at the 178-ft (54.3 m) depth, the results of the experiments at both depths can be combined to provide an extremely well developed base of information regarding the behavior of axial piles in plastic clays.

5.5.1 Experiment 1

Experiment 1 at the 208-ft (63.4 m) depth was performed to investigate the long-term consolidation behavior of the soil, and to obtain values of shear transfer near the end of a long period of undisturbed consolidation.

The experiment was begun on 7 December 1982, with the driving operation being completed at 19:22. A load test which consisted of a large-displacement loading in tension with one cycle of load reversal was performed 7 minutes after driving.

The soil was allowed to consolidate for 4158 minutes, then subjected to eight cycles of reversed large-displacement loading.

The variation in soil pressures during consolidation are shown in Plate 5.64. Again, as for the earlier experiments, the break in the curves corresponds to the time of the load test performed soon after driving. The maximum total pressure at the end of driving was 28.77 ksf (1377 kPa); the accompanying pore pressure was 27.67 ksf (1324 kPa). At the end of the load test which was performed immediately after driving, the total radial pressure was 28.45 ksf (1361 kPa), and the pore pressure was 26.91 ksf (1288 kPa), yielding a value of 1.54 ksf (74 kPa) for the radial effective pressure. During the 69-hour consolidation period, the total radial pressure decreased to 26.76 ksf (1280 kPa) and the pore pressure decreased to 22.59 ksf (1081 kPa), yielding an increase in the radial effective pressure to 4.17 ksf (200 kPa).

During the load test performed immediately after driving, the maximum shear transfer on the initial loading to failure was 0.47 ksf (22 kPa); upon reversal of loading, the shear transfer was reduced to 0.31 ksf (15 kPa). The shear transfer-displacement behavior recorded after the reversal of loading is shown in Plate 5.65.

The results of the large-displacement, two-way loading during the cyclic tests performed after 4158 minutes of consolidation are also contained in Plate 5.65. The peak shear transfer on the first cycle was 0.97 ksf (46 kPa); the post-peak shear transfer, prior to load reversal, was 0.88 ksf (42 kPa). After eight cycles of loading, the cyclic minimum shear transfer had been reduced to 0.63 ksf (30 kPa).

The variations in the pore pressure and the radial effective pressure with the reversals of shear transfer are shown in Plate 5.66. As seen in the plate, the pore pressure increased, with a simultaneous decrease in the radial effective pressure, immediately upon the enforcement of plastic slip in the soil. During the continued reversal of loading, the pore pressure and the effective pressure tended to fluctuate about mean values of each which were above and below the pre-test values, respectively. The abrupt decreases in the pore pressure, and the increases in the radial effective pressure, again occur during periods of plastic slip, at constant values of shear.

At the conclusion of the load test, static pressure measurements showed a value of 26.31 ksf (1259 kPa) for the total pressure, a decrease of 0.45 ksf (22 kPa), and a value of 23.49 ksf (1124 kPa) for the pore pressure, an increase of 0.90 ksf (43 kPa), combining to result in a decrease of 1.35 ksf (65 kPa) in the radial effective pressure, to a value of 2.82 ksf (135 kPa).

During a 10-minute period of rest, the excess pore pressures rapidly dissipated, decreasing to 23.08 ksf (1104 kPa). During the same period, the total pressure remained reasonably constant, being 26.28 ksf (1257 kPa) at the end of 10 minutes.

It can thus be seen that the pattern of response of the soil pressures during this experiment were very similar to those noted for the 58 (17.7), 148 (45.1), and 178-ft (54.3 m) depths. Such similarity indicates that one fundamental mechanism governs the process of cyclic degradation and recovery in resistance.

Since the clay near the pile should retain the same structural arrangement (i.e. no further remolding, such as that which accompanies driving), the changes in shearing resistance must be related to changes in void ratio, and not to losses in shear strength normally associated with the remolding of sensitive clays. The concepts discussed earlier, and the mechanism which was proposed for the process of cyclic degradation thus seems to have been again reinforced during this experiment.

Upon completion of the load test, the probe was moved downward until the mechanical slip-joint was closed. It was then attempted to redrive the probe (with the cutting shoe not yet being broken free) with the 300-lb (1.33 kN) casing hammer. The attempts to drive the probe were unsuccessful. The hydraulic ram was then used to break the cutting shoe free, and was extended to its full stroke upward. The probe was then removed from the boring. Except for the initial downward movement of the probe (not shown), no data was taken during the process.

5.5.2 Experiment 2

The second experiment at the 208-ft (63.4 m) depth was performed with the primary objective of obtaining a value of shear transfer at a low degree of consolidation, so that the increase in the shear transfer with time could be observed. In addition, cyclic load tests were performed late in the consolidation process, followed by an additional long period of time for reconsolidation, with the final series of tests being performed at a near-complete stage of consolidation, so that the process of recovery in shear transfer capability could be observed.

For Experiment 2, a probe was installed by driving on 3 December 1982, at 12:39. The probe was subjected to a large-displacement loading in tension, with one cycle of load reversal, 13 minutes after driving. The probe was then subjected to large-displacement, two-way cyclic tests after 510, 4189, and 8593 minutes of consolidation.

The variation in soil pressures during consolidation are shown in Plate 5.67. The time at which the load tests were performed can be seen in the plate, at points where the curves are discontinuous. The maximum total radial pressure immediately after driving was 28.05 ksf (1342 kPa), with a maximum pore pressure of 23.60 ksf (1129 kPa). As shown in the plate, the total radial pressure and the pore pressure both decreased with time after driving, with the effects of the cyclic load tests being only a temporary disturbance of the consolidation process.

At the end of the load test performed 13 minutes after driving, the total radial pressure was 27.96 ksf (1338 kPa), with an accompanying pore pressure of 23.97 ksf (1147 kPa), for a calculated value of 3.99 ksf (191 kPa) for the radial effective pressure. Prior to the load test performed 510 minutes after driving, the total radial pressure was 26.79 ksf (1282 kPa) and the pore pressure was 22.92 ksf (1097 kPa), for a value of 3.87 ksf (185 kPa) for the radial effective pressure.

At the time of the cyclic load tests performed 4189 minutes after driving, the total radial pressure had decreased to 26.22 ksf (1255 kPa) and the pore pressure had decreased to 21.79 ksf (1043 kPa), for a calculated value of 4.43 ksf (212 kPa) for the radial effective pressure.

At the time of the final load test, performed 8593 minutes after driving, the total radial pressure was 26.33 ksf (1260 kPa) and the pore pressure was 21.73 ksf (1040 kPa), for a calculated value of 4.60 ksf (220 kPa) for the radial effective pressure.

During the load test performed 13 minutes after driving (not shown), the peak shear transfer on the initial loading to failure was 0.55 ksf (26 kPa), which reduced to 0.45 ksf (22 kPa) on the second failure in tension, with a residual, or post-peak, value of 0.34 ksf (16 kPa) at large displacements.

The results of the large-displacement, two-way cyclic tests performed after 510 minutes of consolidation are given in Plate 5.68, which contains the shear transfer-displacement behavior during the tensile loading to failure on the first and the ninth cycles of loading. The peak shear transfer on the first cycle of loading was 0.82 ksf (39 kPa), with a reduced post-peak value of 0.75 ksf (36 kPa); the peak shear transfer was reduced to 0.66 ksf (32 kPa) on the ninth cycle, with a constant post-peak value of 0.60 ksf (29 kPa).

A comparison of Experiments 1 and 2 indicates that, again, the shear transfer capacity of the soil increased rapidly with time after driving, being 0.83 ksf (40 kPa) after 510 minutes in this experiment, and 0.98 ksf (47 kPa) after 4158 minutes in the first experiment. It is also of interest to note that the value of cyclic minimum shear transfer in this experiment, 0.60 ksf (29 kPa), was very close to the value of 0.63 ksf (30 kPa) measured in Experiment 1.

The variations in the pore pressure, the radial effective pressure, and the shear transfer during the cyclic load tests are given in Plate 5.69. As shown in the plate, the pore pressures were generally elevated during the cyclic tests, with the radial effective pressures being reduced. The abrupt decreases in pore pressure, and the accompanying increases in the radial effective pressure, occur during plastic slip in compression. Only small changes occurred during the tensile loading.

At the end of the cyclic tests, static pressure measurements indicated a value of total pressure of 27.00 ksf (1292 kPa) and a value of pore pressure of 23.13 ksf (1107 kPa), yielding a value of 3.88 ksf (185 kPa) for the radial effective pressure. Thus, during this load test, the total pressure and the pore pressure both increased by 0.21 ksf (10 kPa). After one hour, the excess pore pressures had returned to the values recorded prior to the test. No intermediate pressure measurements were made, so that the early rate of dissipation is not known.During a period of time allowed for reconsolidation, the total pressure decreased to 26.22 ksf (1255 kPa) and the pore pressure decreased to 21.79 ksf (1043 kPa), resulting in an increase in the radial effective pressure to 4.43 ksf (212 kPa).

The results of the large-displacement, two-way cyclic tests performed after 4189 minutes of consolidation are given in Plate 5.70, which shows the shear transfer-displacement behavior during the tensile loading to failure on the first and the fifth cycles of loading. The peak shear transfer on the first cycle of loading was 1.19 ksf (57 kPa), with the post-peak resistance still decreasing at the time the loading was reversed; the peak shear transfer was reduced to 0.85 ksf (41 kPa) on the fifth cycle, with the post-peak, or residual, value being 0.80 ksf (38 kPa).

The increase in shear transfer from 0.60 ksf (29 kPa) at the end of the first series of cyclic tests to 1.19 ksf (57 kPa) indicates that the soil had completely recovered from the earlier cyclic degradation, and had even increased beyond the value of 0.98 ksf (47 kPa) which had been recorded during Experiment 1 at a similar time after installation. The cyclic minimum resistance also increased from 0.60 (29) to 0.80 ksf (38 kPa). Thus, the dissipation of excess pore pressures from both the installation and the cyclic loading resulted in an increase in the initial and the cyclic minimum shear transfer to values that were larger than those obtained after an equal period of undisturbed consolidation.

The variations in the pore pressure, the radial effective pressure, and the shear transfer during the cyclic load tests are shown in Plate 5.71. Again, the abrupt decreases in the pore pressures occur during plastic slip, with larger decreases during compressive loading, and the smaller decreases occurring during the upward plastic slip.

At the end of the cyclic load tests, the static pressure measurements showed a value of 26.11 ksf (1249 kPa) for the total pressure, a decrease of 0.11 ksf (5 kPa); the pore pressure was 22.19 ksf (1062 kPa), an increase of 0.40 ksf (19 kPa). During the first hour after the load test, the total pressure slowly decreased, to a value of 26.00 ksf (1244 kPa); at the same time, the pore pressure increased, to a peak value of 22.36 ksf (1070 kPa), whereupon the pore pressure again began decreasing.

Again, this pattern of pore pressure response suggests an initial flow of pore water into the shear zone, alleviating a gradient in pore pressure toward the failure surface. Once the pressure deficiency in the shear zone has been relieved, the flow of pore water is again outward, relieving the excess pore pressures in the clay near the pile.

A period of 17 hours was required for the excess pore pressure to dissipate to the pre-test level after these tests, as compared with one hour after the first series of tests. If the time required for the excess pore pressures in the clay between the slip surface and the pile to dissipate is a function of the thickness of the layer which consolidates (or distance to the surface of slip), then the radial distance to the shear zone in these tests was approximately 4 times the radial distance to the slip surface which was formed during the first cyclic tests.

The magnitude of shear transfer is a function of both the radial distance to the shear surface and the shear stress on the surface. A comparison of the cyclic minimum shear transfer measured during the two experiments indicates that either an increase in shear strength, or an increase in the radial distance to the slip surface, has occurred. Perhaps both effects occurred simultaneously.

An increase in the shear strength of the clay within the initial slip zone to a value only slightly greater than that which previously existed will result in an outward shift of the failure surface, since the radial variation of shear stress during loading is unchanged. It is not expected that all of the 33 percent increase in the cyclic minimum shear transfer are due to an increase in the radial distance to the failure surface. However, at least half of the increase could be easily due to such an occurrence.

An additional period of 4400 minutes was then allowed for reconsolidation (for a total consolidation time of 8593 minutes). During this period, the total pressure showed a slight increase, to 26.33 ksf (1260 kPa), and the pore pressure decreased to 21.73 ksf (1040 kPa), combining to indicate an increase in the radial effective pressure to 4.60 ksf (220 kPa).

The results of the large-displacement, two-way cyclic tests performed after 8593 minutes of consolidation are given in Plate 5.72, which shows the shear transfer-displacement behavior during the tensile loading to failure on the first and the tenth cycles of loading. The peak shear transfer on the first cycle of loading was 1.04 ksf (50 kPa); the limiting shear transfer on the tenth cycle was 0.80 ksf (38 kPa).

A comparison of the results of these tests with the second series of tests indicates an incomplete recovery in resistance. The cyclic minimum resistance remained the same, while the maximum shear transfer of 1.04 ksf (50 kPa) was less than the value of 1.19 ksf (57 kPa) recorded earlier. It should be noted, however, that the actual increase in resistance was from 0.80 (38) to 1.04 ksf (50 kPa); which does indicate a significant degree of recovery.

Since the magnitude of the cyclic minimum shear transfer is the same, it is likely that failure occurred on or near the same failure surface that had been created in the second series of cyclic tests, with the clay adjacent to the surface having almost the same void ratio as before.

The variations in pore pressure, radial effective pressure, and shear transfer during these tests are given in Plate 5.73. Again, the abrupt decreases in pore pressure (and simultaneous increases in the radial effective pressure) occur during plastic slip in compression. Only minor changes were observed during upward plastic slip.

Following the cyclic load tests, the probe was subjected to an additional large-displacement loading, with one cycle of load reversal. During the final loading in tension, shown in Plate 5.74, the loading rate was increased from 0.001 inch (0.025 mm)/sec to 0.010 inch (0.25 mm)/sec. In response to the ten-fold increase in slip rate, the shear transfer increased from 0.77 ksf (37 kPa) to 0.86 ksf (41 kPa), an increase of almost 12 percent.

The simultaneous variation with time in the pore pressure, the radial effective pressure, and the shear transfer during the variable-rate test are given in Plate 5.75. As seen in the plate, the effect of plastic slip in the soil is an increase in the radial effective pressure and a decrease in the pore pressure. During the second loading in tension, the magnitudes of the changes in the pressures can be seen to be even greater than during the slow-rate failure. During the fast-rate test, the pore pressure decreased to a minimum value of 20.56 ksf (984 kPa), with a value of total pressure of 26.09 ksf (1248 kPa) being recorded, yielding a value of 5.53 ksf (265 kPa) for the instantaneous value of radial effective pressure during the test.

After the end of the fast-rate test, the static pressure measurements indicated a total pressure of 26.48 ksf (1267 kPa) and a pore pressure of 21.34 ksf (1021 kPa), for a radial effective pressure of 5.14 ksf (246 kPa).

Following this test, the probe was monitored for a period of 25 hours. During this period, the pore pressures rapidly increased, to a maximum value of 21.90 ksf (1048 kPa) after 5 minutes, and then slowly decayed to a minimum value of 21.55 ksf (1031 kPa) after 18 hours. The total pressures during the same period showed a slow decrease, with minor fluctuations, ending with a value of 26.14 ksf (1251 kPa) after 25 hours.

Again, the pattern of pore pressure response after a cyclic test has suggested a period of inward flow of pore water toward the failure surface, relieving a pressure deficiency in the shear zone, followed by a dissipation of the excess pore pressures due to volume changes that the inward flow of pore water has produced.

At this time, the probe was removed from the boring without a load test. Once the probe had been removed from the boring, the surface of the probe was carefully examined. As the film of clay which had adhered to the probe was removed, two distinct slip surfaces became apparent, with the clay layers having an appearance much like that of an onion.

The formation of successive shear zones in the soil, with each being slightly larger in diameter than the last, suggests that a part of the recovery in shear transfer capacity after cyclic degradation may be due to an increase in the area of the cylindrical shear surface, and not necessarily be due to increases in shearing resistance in the soil. Since consolidation, and the decrease in void ratio of the clay, must accompany the outward flow of pore water, some portion of the recovery in resistance is due to an increase in shear strength However, since the effective shear area is also increased, the portions arising from each source may be difficult to identify.

As noted earlier, the time-rate of dissipation of the excess pore pressures after the load tests also suggested that multiple shear zones had been developed, as did the increase in the cyclic minimum shear transfer. Thus, all the observations tended to confirm the concept of the outward movement of a shear zone. The final proof lay in the inspection of the probe after its removal from the boring.

5.5.3 Experiment 3

The primary objective of the third experiment at the 208-ft (63.4 m) depth was to examine the behavior under small displacement cyclic loading. A period of consolidation similar to that allowed in Experiment 2 was used, in order to compare the magnitude of shear transfer at which cyclic degradation was initiated to that obtained when the probe was immediately loaded to failure. In addition, a short period of time was allowed for reconsolidation, in order to observe the increase in resistance after cyclic degradation.

For this experiment, a probe was installed on 9 December 1982, with the driving operation being completed at 13:24. A large-displacement test with one cycle of load reversal was performed 6 minutes after the end of driving. The soil was allowed to consolidate for 477 minutes, at which time a sequence of progressively-increased, symmetric two-way displacement-controlled tests were performed. Upon completion of the cyclic tests, the soil was allowed to continue to consolidate until 1122 minutes had elapsed since driving, at which time the probe was subjected to a loading to failure in tension, followed by one cycle of load reversal.

The variation in soil pressures during consolidation are shown in Plate 5.76. The effects of the load tests on the static soil pressures can be seen in Plate 5.76, in which the curves are broken at the time of, and for the duration of, each series of load tests. The maximum total pressure immediately after driving was 29.92 ksf (1432 kPa), with an accompanying pore pressure of 29.54 ksf (1413 kPa). At the end of the load test performed 6 minutes after the end of driving, the total radial pressure was 29.65 ksf (1419 kPa) and the pore pressure was 28.26 ksf (1352 kPa), yielding a calculated value of radial effective pressure of 1.39 ksf (67 kPa).

After 477 minutes of consolidation, prior to the cyclic load tests, the total radial pressure had decreased to 28.88 ksf (1382 kPa), and the pore pressure had decreased to 25.04 ksf (1198 kPa), indicating an increase in the radial effective pressure to 3.84 ksf (184 kPa). Upon completion of the cyclic load tests, the total radial pressure was 28.32 ksf (1355 kPa) and the pore pressure was 25.46 ksf (1218 kPa), yielding a value of 2.86 ksf (137 kPa) for the radial effective pressure.

After 1122 minutes of consolidation, prior to the final load test, the total radial pressure had decreased to 28.25 ksf (1352 kPa) and the pore pressure had decreased to 24.47 ksf (1171 kPa), yielding a calculated value of 3.78 ksf (181 kPa) for the radial effective pressure.

The variation in the shear transfer, the pore pressure, and the radial effective pressure during the small-displacement cyclic tests are shown in Plate 5.77. The behavior during these tests was much like that shown in Plate 5.10 for similar tests at the 58-ft (17.7 m) depth. Again, the average pore pressure increased during the initial loading of the probe, then fluctuated about an elevated value throughout the cyclic tests. As the limits of displacement were increased, the average pore pressure slowly increased. In each cycle of loading, the pattern of the pore pressure variation was the same as had been observed earlier; i.e., decreases at the peak load, with a return to the average value during the reversal of load.

During the load test performed 6 minutes after driving, the peak shear transfer on the initial failure was 0.42 ksf (20 kPa); after one reversal of loading, the shear transfer was reduced to 0.27 ksf (13 kPa). The shear transfer-displacement behavior recorded after the reversal of loading is given in Plate 5.78.

As seen in Plate 5.77, cyclic degradation was occurring throughout the test, with a decrease in the peak resistance noted for successive cycles of loading between each set of progressively-increased displacement limits. The maximum value of shear transfer was recorded on the first excursion to a displacement of 0.025 inch (0.63 mm), and was 0.74 ksf (35 kPa).

Upon completion of the tests with progressively-increased displacements, the probe was subjected to six cycles of large-displacement, two-way loading. The results of these tests are given in Plate 5.78, which shows the shear transfer-displacement behavior during tension loading on the first and sixth cycle of loading. The maximum shear transfer on the first cycle was 0.72 ksf (34 kPa), near the value of 0.74 ksf (35 kPa) which had been recorded during the earlier tests. After five cycles of loading, the shear transfer had stabilized at a value of 0.57 ksf (27 kPa).

A comparison of the results of this experiment with those from Experiment 2 at a similar time after driving indicates that cyclic degradation had indeed begun at a value of shear transfer which was smaller than the maximum shear transfer which would be expected at this stage of consolidation, near 0.8 ksf (38 kPa), but was greater than the cyclic minimum resistance, which was 0.57 ksf (27 kPa) in this experiment and 0.60 ksf (29 kPa) in the second.

Again, the pattern of the pore pressure response during cyclic loading suggests that the mechanism of cyclic degradation includes the flow of pore water into the shear zone during active shear, with a resulting increase in the void ratio, and subsequent loss of resistance. As the pore water is drawn into the shear zone, the pore pressure in the clay between the failure surface and the pile slowly increases, as the shear zone expands to accommodate the added pore water.

The effects of the cyclic tests on the static soil pressures measured at the end of the test indicated a decrease in the total pressure to 28.32 ksf (1355 kPa) and an increase in the pore pressure to 25.46 ksf (1218 kPa). The pore pressure then fluctuated for a period of time, first decreasing to a minimum value of 25.30 ksf (1211 kPa) after 15 minutes, then generally increasing, also being 25.43 ksf (1217 kPa) at the end of one hour. The excess pore pressures then rapidly dissipated, reaching the value measured prior to the cyclic tests in only 30 additional minutes (1.5 hours after the end of loading).

Again, this response indicates that the instantaneous value of pore pressure in the clay near the slip surface was much lower than at the pile wall; upon the cessation of slip, the pore pressure at the pile wall continued to decrease, until a minimum value was reached. Then, the pore pressure increased as flow toward the failure surface relieved the deficiency, whereupon the entrapped excess pore pressures began to dissipate.

After the end of the cyclic tests, a period of 6.5 hours (for a total consolidation time of approximately 18.7 hours) was allowed for reconsolidation and recovery in shear transfer capacity in the soil. At the end of this period, the total pressure decreased to 28.25 ksf (1352 kPa), accompanied by a decrease in the pore pressure to 24.47 ksf (1171 kPa), which combine to result in an increase in the radial effective pressure to 3.78 ksf (181 kPa), near the value which was recorded prior to the cyclic tests.

The results of the load tests performed after 1122 minutes of consolidation are given in Plate 5.79 and 5.80. Plate 5.79 shows the initial tensile loading to failure. The peak shear transfer on the initial failure in tension was 0.80 ksf (38 kPa); after one reversal, the peak was reduced to 0.73 ksf (35 kPa), with a reduced post-peak value of 0.66 ksf (32 kPa). The variations in pore pressure, radial effective pressure, and shear transfer recorded during the test are shown in Plate 5.80.

Again, the soil exhibited a rapid recovery in the shearing resistance which had been reduced during the cyclic load tests. From a value of 0.57 ksf (27 kPa), the shear transfer increased to 0.80 ksf (38 kPa), near the value which would be expected in the absence of the intermediate degradation, based on the results of Experiment 2.

Thus, both the cyclic degradation and the recovery in shear transfer capacity again seem to be related to the flow of water into and away from the shear zone. The flow of pore water into a thin layer of clay increases the void ratio, and the moisture content, temporarily reducing the shearing resistance. Upon the cessation of loading, the pore water which was drawn into the shear zone is expelled, resulting in a recovery in resistance.

Upon completion of the load test, soil pressures were monitored closely for the time the loading system was being removed. During the first minute, the pore pressures decreased, from a value of 24.62 ksf (1178 kPa) to a minimum value of 24.22 ksf (1159 kPa). The pore pressures then increased to a peak value of 25.15 ksf (1203 kPa), after four minutes. During the next three minutes, the pore pressure decreased to 24.97 ksf (1195 kPa). At this time, the probe was removed from the boring, with no further observations of the soil pressures being made.

The load test was stopped at the end of a large-displacement, upward loading of the probe. The early decrease in the pore pressure is, again, a delay in the change in pressure at the pile wall due to a greater change in pressure in the shear zone. After the minimum pressure is reached, inward flow of pore water again alleviates the pressure deficiency, then outward flow commences.

5.5.4 Experiment 4

Experiment 4 at the 208-ft (63.4 m) depth was intended to be an examination of the effects of long-term consolidation on the shear transfer response of the clay after cyclic degradation had occurred early in the consolidation history. As noted earlier, moisture intruded into the connector at the end of the cable approximately 21 hours after driving. However, the data which was obtained for the probe prior to the development of leakage was not affected.

For this experiment, a probe was installed on 18 December 1983, with the driving operation being completed at 13:55. The probe was subjected to one large-displacement loading with one cycle of reversal 9 minutes after driving. The soil was allowed to consolidate for 477 minutes, at which time the probe was subjected to ten cycles of large-displacement, two-way loading. The probe was then left in place until April, 1984.

The variations in soil pressure during consolidation are given in Plate 5.81. The total radial pressure immediately after driving was 26.31 ksf (1259 kPa), which increased to 26.50 ksf (1268 kPa) five minutes later. The corresponding pore pressures were 26.39 (1263) and 26.45 ksf (1266 kPa), indicating very low values of radial effective pressure.

At the end of the load test begun 9 minutes after driving, the total radial pressure was 26.25 ksf (1256 kPa), with an accompanying pore pressure of 26.19 ksf (1253 kPa). At the time of the second load test, begun 477 minutes after driving, the total pressure had decreased to 24.89 ksf (1191 kPa) and the pore pressure had decreased to 23.98 ksf (1147 kPa), indicating a value of 0.91 ksf (44 kPa) for the radial effective pressure.

The behavior of the soil pressures during this experiment were similar to those observed during Experiment 4 at the 148-ft (45.1 m) depth. Again, both the total and the pore pressures increased rapidly with time shortly after driving, then began to decrease, as expected.

The last reliable values of pressure were recorded 963 minutes after driving. At this time, the total radial pressure was 24.41 ksf (1168 kPa) and the pore pressure was 22.85 ksf (1093 kPa), indicating a value of 1.56 ksf (75 kPa) for the radial effective pressure.

During the load test performed 9 minutes after driving, the peak shear transfer on the initial failure in tension was 0.24 ksf (11 kPa), with a value of 0.14 ksf (7 kPa) being recorded after the load reversal. The portion of the shear transfer-displacement response which was recorded after the reversal of load is given in Plate 5.82.

The results of the load tests performed after 477 minutes of consolidation are given in Plate 5.82. The plate contains the tension-loading portions of the first and tenth cycles of loading. On the first cycle, the maximum shear transfer was 0.46 ksf (22 kPa), with a value of 0.36 ksf (18 kPa) being measured on the tenth cycle.

The variation of pore pressure, effective pressure and shear transfer during the cyclic test at the 208-ft (64.3 m) depth are shown in Plate 5.83. These show behavior similar to that recorded in previous experiments, with a general increase in pore pressure and associated decease in effective pressure as the cyclic test progresses.

The extremely low values of shear transfer, combined with the abnormal variations in the total and pore pressures immediately after driving, render the results of this particular experiment suspect. In view of the generally good agreement which was obtained among the remaining experiments at this depth, the results of this experiment were presented only for purposes of completeness.

5.5.5 Experiment 5

For the fifth experiment at the 208-ft (64.3 m) depth, a 3-inch (7.62 cm)-diameter probe was installed on 17 December 1983, with the driving operation being completed at 00:36. The probe was subjected to a large-displacement loading, with one reversal of loading to failure 12 minutes after the end of driving. The soil was allowed to consolidate for 4352 minutes, then subjected to ten cycles of two-way, large-displacement loading.

The soil was then allowed to consolidate until 103 days had elapsed since installation (148,766 minutes). The probe was then subjected to ten cycles of two-way, large-displacement cyclic loading. At the end of the ten cycles of loading, a number of additional load cycles were applied, during which the loading rate was varied through the widest range possible, within the limitations of the hydraulic system used.

During the long-term consolidation of the probe, drift was noted in the strain-gage bridge of the total pressure transducer. Readings made in air following removal of the probe showed that the transducer zero reading had shifted by 0.000277 volts, which is equivalent to a radial pressure of 2.58 ksf (124 kPa). Similar comparisons for the pore pressure transducer and the axial load cells showed differences of 0.000039 and 0.000091 volts, respectively, which are equivalent to values of pore pressure of 0.22 ksf (11 kPa) and of shear transfer of 0.04 ksf (2 kPa).

During the reduction of the raw data, the in-air zero readings made prior to deploying the probe were used to calculate engineering units from the data from the experiment in 1983. Those made after removing the probe were used for the data taken in 1984. Thus, the early part of the consolidation data are probably correct, as are the values reported for the load tests. Since the variation of the zero readings with time are not known, the values of pressure reported for the dates between December 1983 and 28 March 1984 are likely to be in error.

The variation in the soil pressures during consolidation are given in Plate 5.84. The maximum radial total pressure, recorded immediately after driving, was 30.86 ksf (1477 kPa). The maximum pore pressure, recorded a short time later, was 29.47 ksf (1410 kPa). At the end of the load test performed immediately after driving, the radial total pressure was 30.33 ksf (1451 kPa) and the pore pressure was 29.34 ksf (1404 kPa), indicating a value of 0.99 ksf (47 kPa) for the radial effective pressure.

Both the total and the pore pressure decreased during a 4352-minute period of undisturbed consolidation. After 4352 minutes, the radial total pressure was

27.17 ksf (1300 kPa) and the pore pressure was 22.57 ksf (1080 kPa), indicating a value of 4.60 ksf (220 kPa) for the radial effective pressure.

Upon completion of the large-displacement cyclic tests, the total pressure was recorded as 26.84 ksf (1284 kPa), with an accompanying value of pore pressure of 24.62 ksf (1178 kPa), yielding a reduction in the radial effective pressure to 2.22 ksf (106 kPa).

As shown in Plate 5.84, the pore pressure decreased continuously during the next 103 days, with a value of 21.59 ksf (1033 kPa) being recorded on 28 March. The total pressure at the same time was 25.61 ksf (1225 kPa), yielding a value of 3.89 ksf (186 kPa) for the radial effective pressure.
The maximum shear transfer recorded during the load test performed immediately after driving was 0.10 ksf (5 kPa), with a reduced value of 0.08 ksf (4 kPa) being recorded after one reversal of loading, as shown in Plate 5.85.

The results of the final cycle of the load tests performed after 4352 minutes of consolidation are given in Plate 5.85. During this load test, the analog and digital recorders were inadvertently set to monitor a different probe; the load to failure in tension during the first cycle was therefore lost. The maximum shear transfer measured during the first cycle of loading in compression was 1.00 ksf (48 kPa). Plate 5.85 shows the shear transfer-displacement behavior recorded during the loading in tension on the tenth cycle. The peak shear transfer recorded on the tenth cycle was 0.82 ksf (39 kPa); the reduced post-peak value was 0.78 ksf (37 kPa).

The variation in the pore pressure, radial effective pressure, and shear transfer during this series of cyclic tests are shown in Plate 5.86. As seen in the plate, the initial loading to failure of the probe (which was not recorded) resulted in a decrease in the radial effective pressure and an increase in the pore pressure, much as had been observed in earlier experiments. The pressures then fluctuated above and below a reasonably-constant mean value, with the simultaneous decreases in pore pressure and increases in the effective pressure again occurring during constant-shear plastic slip.

At the end of the cyclic tests, two cycles of loading (not shown) were applied during which the load rate was varied from 0.0008 inch (0.020 mm)/sec to 0.029 inch (0.74 mm)/sec. At the slow rate, the recorded value of shear transfer was 0.69 ksf (33 kPa); at the faster rate, the shear transfer increased to 1.04 ksf (50 kPa).

After the completion of the fast-rate tests, the value of total pressure which was recorded was 26.84 ksf (1284 kPa), with an accompanying value of pore pressure of 24.62 ksf (1178 kPa), yielding a value of 2.22 ksf (106 kPa) for the radial effective pressure. The probe was then monitored for three hours, during which time the total pressure decreased to 26.46 ksf (1266 kPa) and the pore pressure decreased to 22.93 ksf (1097 kPa), indicating an increase in the radial effective pressure to 3.53 ksf (169 kPa).

As noted earlier, the pressures measured just prior to performing the long-term load test on 29 March, 1984, indicated a total pressure of 25.61 ksf (1225 kPa) and a pore pressure of 21.59 ksf (1033 kPa), for a value of radial effective pressure of 4.02 ksf (192 kPa).

The results of the two-way cyclic tests to large displacements performed after 103 days of consolidation are given in Plate 5.87. The plate shows the shear transfer-displacement behavior recorded during tension loading in the first cycle and during the final slow loading to failure prior to removing the probe. The peak shear transfer on the first cycle was 1.41 ksf (68 kPa), with a minimum post-peak value of 1.08 ksf (52 kPa) being recorded, although the shear transfer was still decreasing with increasing displacement. After ten cycles, the peak shear transfer reduced to 1.05 ksf (50 kPa), with a post-peak value of 0.94 ksf (45 kPa). As shown in the plate, the resistance decreased still further during additional cyclic loading, showing a peak resistance of 1.01 ksf (48 kPa) and a post-peak residual value of 0.85 ksf (41 kPa) after six additional load cycles performed at various load rates.

The results of this experiment clearly indicates that, following long-term consolidation after cyclic degradation, the shear transfer which was attained was higher than would be expected under undisturbed consolidation.

The variations in the pore pressure, radial effective pressure, and shear transfer during the cyclic test are shown in Plate 5.88. As seen in the plate, the pore pressure increased slightly during the initial tensile loading, with the first abrupt decrease in the pore pressure accompanying the plastic slip after yield. After the first cycle, the pore pressure decreases coincide with plastic slip in the soil. The decreases were larger during the tensile-loading part of the cycle than for the compressive loading.

Following the last cycle of the two-way cyclic test, the loading rate was increased, and four cycles of loading were applied. The last cycle of two-way loading was performed at a rate of displacement of 0.0007 inch (0.018 mm)/sec, with a value of shear transfer of 0.94 ksf (45 kPa); at a loading rate of 0.031 inch (0.79 mm)/sec, the shear transfer was increased to 1.14 ksf (55 kPa).

The effects of the variation in load rate on the soil pressures are shown in Plate 5.89. As seen in the plate, the abrupt decreases in pore pressure (and simultaneous increases in the radial effective pressure) accompany plastic slip in tension. Only minor effects were noted during compressive loading.

Upon completion of the fast-rate cyclic tests, the probe was subjected to two additional loadings to failure in tension, with no loading to failure in compression. The initial loading to failure was performed with a rate of plastic slip of 0.006 inch (0.15 mm)/sec; the second at a rate of 0.029 inch (0.74 mm)/sec. The results of these final loadings are shown in Plate 5.90. As seen in the plate, the increase in the rate of slip from 0.006 inch (0.15 mm)/sec to 0.029 inch (0.74 mm)/sec resulted in an increase in the limiting shear transfer from 0.85 ksf (41 kPa) to 1.13 ksf (54 kPa), an increase of 33 percent.

Upon completion of these tests, the probe was left in place for 4.5 hours, then removed from the boring. During the first minute after loading, the pore pressure increased from 22.74 (1088) to 22.78 ksf (1090 kPa); thereafter, the pore pressures decreased, reaching 21.77 ksf (1042 kPa) after a period of 3.3 hours, a value which was still higher than the value measured prior to the load test, 21.59 ksf (1033 kPa).

Again, the slower rate of dissipation of the excess pore pressures after the conclusion of the cyclic loading suggests that the radial distance to the shear zone may be larger in this experiment than those observed in the other experiments at this depth, with the possible exception of Experiment 2. Thus, a portion of the increase in shear transfer may be related to the distance to the surface of failure; however, the predominant effect is an increase in the shear strength.

5.5.6 Experiment 6

The sixth experiment at the 208-ft (63.4 m) depth was performed in order to observe the consolidation behavior over a long period of time, and to obtain a value of ultimate shear transfer after the completion of undisturbed consolidation.

For this experiment, a 3-inch (7.62 cm)-diameter probe was installed on 17 December 1983, with the driving operation being completed at 16:37. The probe was then tested at large displacements, with one cycle of reversal, 9 minutes after the end of driving. The soil was then allowed to consolidate for 103 days, then subjected to ten cycles of two-way, large-displacement loading.

As noted earlier for the probe used in Experiment 5, a zero-shift was also observed for the total pressure transducer in the probe used in this experiment. The in-air zero pressure readings made after the probe was removed differed from those made in air prior to deploying the probe by 0.000158 volts, which corresponds to a value of total pressure of 1.47 ksf (70 kPa).

Comparisons of the two sets of zero-pressure voltage readings for the pore pressure transducer showed a difference of 0.000039 volts, equivalent to 0.12 ksf (6 kPa). For the axial load cells, the difference was 0.000040 volts, which is equivalent to 0.02 ksf (1 kPa) when converted to shear transfer.

As noted earlier for Experiment 5, the in-air zero pressure readings made in December prior to deploying the probe were used to deduce the pressures measured in 1983; those made after removal of the probe in 1984 were used when reducing the data taken at that time.

The variation in soil pressures during consolidation are shown in Plate 5.91. The maximum radial total pressure, recorded 5 minutes after driving, was 28.45 ksf (1361 kPa). The maximum pore pressure, recorded 4 minutes after driving, was 28.57 ksf (1367 kPa), which indicated very low values of radial effective pressure.

During the load test performed 8 minutes after driving, the peak shear transfer on the initial failure in tension (not shown) was 0.31 ksf (15 kPa); after one reversal of plastic slip, the shear transfer was reduced to 0.23 ksf (11 kPa), as shown in Plate 5.92.

The shear transfer versus displacement relationships recorded during the first loading to failure and the tenth cycle of loading after 2467 hours 9 minutes of consolidation are also shown in Plate 5.93. On the first loading, the peak shear transfer was 1.06 ksf (51 kPa), which reduced to 0.82 ksf (39 kPa) at the time the loading was reversed. On the tenth cycle, the peak shear transfer had decreased to 0.74 ksf (35 kPa), with a post-peak residual value of 0.65 ksf (31 kPa).

The simultaneous variations in the shear transfer, the pore pressure, and the radial effective pressure, are shown in Plate 5.93. As seen in the plate, the pore pressure increased during the initial loading to failure, with a corresponding decrease in the radial effective pressure. During the subsequent cycles of loading, the pressures fluctuated above and below values of pressure which were created by the shearing on the first cycle of loading.

Upon completion of the primary series of cyclic tests, the probe was subjected to five cycles of fast-rate cyclic loading, followed by one slow-rate cycle. The tension-loading portions of the fifth fast-rate and the slow-rate cycle are shown in Plate 5.94. As seen in the plate, the increase in slip rate from 0.001 inch (0.025 mm)/sec to 0.021 inch (0.53 mm)/sec resulted in an increase in the shear transfer from 0.60 (29) to 0.79 ksf (38 kPa), an increase of approximately 32 percent.

No effects of rate on the slope of the shear transfer-displacement response were recorded; however, it should be noted that, during the time that the load on the probe is increasing, the string of N-rods accumulates a considerable amount of elastic deformation. Thus, the rate at which the probe displaces during the increase in load is considerably smaller than the rate of displacement during plastic slip, when the shear is relatively constant, with a constant amount of elastic deformation in the string of N-rods. The rate-effect tests can thus be used to determine the effects of slip rate on the maximum shear transfer, but cannot be used to infer any effects of rate on stiffness.

The variation in the soil pressures and the shear transfer during the five cycles of fast-rate loading and the slow-rate cycle are shown in Plate 5.95. As seen in the plate, the decreases in pore pressure coincide with periods of plastic slip in the soil. At the same time, the average value of pore pressure increases. Again, this suggests that pore water is being drawn into the shear zone during plastic slip, and, as a result, increases the net pore pressure in the clay which lies between the pile wall and the slip surface.

After completion of the slow-rate cyclic test, the probe was subjected to one additional large-displacement loading in tension. The ram was then extended for the full stroke to break the cutting shoe free. The probe was then monitored for a period of 5 hours, then removed from the boring.

The results of the final loading of the probe are shown in Plate 5.96. As shown in the plate, the rate of displacement of the probe was increased during plastic slip from a nominal rate of 0.001 inch (0.025 mm)/sec to 0.021 inch (0.53 mm)/sec. As shown in the plate, the resistance of the soil increased from 0.60 ksf (29 kPa) at the slow rate to 0.80 ksf (38 kPa) at the faster rate, much the same increase as had been observed in the earlier cyclic tests.

It is of interest to note that the value of cyclic minimum resistance, 0.60 ksf (29 kPa), is the very nearly the same value or that which was observed in Experiments 2 and 3 after approximately 8 hours of undisturbed consolidation, and in Experiment 1 after 69 hours of undisturbed consolidation.

The variations in the shear transfer and the soil pressures during this test are shown in Plate 5.97. As seen in the plate, the pore pressure initially decreased during the slow loading, then began to increase. When the rate of loading was increased, the pore pressures initially increased, then dropped again. Since the probe was momentarily motionless, as the rate was changed, the pore pressures indicate a rapid increase. Near the end of the load test, the rate of displacement was again decreased, to the slow rate. As soon as the rate is decreased, the pore pressures begin a rapid increase, finally becoming constant near the end of the test.

At the end of the load test, the pore pressure was 24.22 ksf (1159 kPa), an increase of 2.48 ksf (119 kPa) above the value of 21.77 ksf (1042 kPa) which had been recorded prior to the load tests. The excess pore pressures then rapidly dissipated, reaching a value of 22.65 ksf (1084 kPa) after one hour, and a minimum value of 22.31 ksf (1068 kPa) after a period of four hours.

5.5.7 Summary of Results at the 208-ft (63.4 m) Depth

The results of the experiments at the 208-ft (63.4 m) depth are tabulated below. The periods of elapsed time in the table are those since the end of driving (See Table 4.4, Section 4.3). The soil pressures are, again, taken from static measurements prior to, and after the completion of, the cyclic load tests.

TABLE 5.4 RESULTS OF THE EXPERIMENTS AT THE 208-FT (63.4 M) DEPTH

Experiment Number	Date	Time	Elapsed Time (min)	Soil Pressures Total ksf	Pore (kPa)	Shear Transfer / Cycle Num ksf (kPa)
1	07 Dec 1982	19:29	7	28.47 (1362)	27.23 (1303)	0.47 / 1 (22)
	10 Dec 1982	16:40	4158	26.76 (1280)	22.59 (1081)	0.97 / 1 (46)
		17:21	4199	26.31 (1259)	23.49 (1124)	0.63 / 8 (30)
2	03 Dec 1982	12:52	13	27.90 (1335)	23.75 (1136)	0.55 / 1 (26)
		21:09	510	26.79 (1282)	22.92 (1097)	0.82 / 1 (39)
		21:49	550	27.00 (1292)	23.13 (1107)	0.66 / 9 (32)
	06 Dec 1982	10:28	4189	26.22 (1255)	21.79 (1043)	1.19 / 1 (57)
		11:10	4231	26.11 (1249)	22.19 (1062)	0.85 / 5 (41)
	09 Dec 1982	11:52	8593	26.33 (1260)	21.73 (1040)	1.04 / 1 (50)
		12:40	8641	26.38	22.06	0.80 / 10

				(1262)	(1056)	(38)
3	09 Dec 1982	13:30	6	29.92 (1432)	28.87 (1381)	0.42 / 1 (20)
		21:21	477	28.88 (1382)	25.04 (1198)	N/A
	10 Dec 1982	01:12	708	28.32 (1355)	25.46 (1218)	0.57 / 86 (27)
		07:56	1122	28.25 (1352)	24.47 (1171)	0.80 (38)
4	18 Dec 1983	14:04	9	26.45 (1266)	26.44 (1265)	0.24 / 1 (11)
		21:52	477	24.89 (1191)	23.98 (1147)	0.46 / 1 (22)
		23:30	575	24.79 (1186)	24.04 (1150)	0.36 / 10 (17)
5	17 Dec 1983	00:38	12	30.26 (1448)	29.44 (1409)	0.10 / 1 (5)
	20 Dec 1983	00:58	4352	27.17 (1300)	22.57 (1080)	N/A
		02:39	4513	26.84 (1284)	24.62 (1178)	0.82 / 10 (39)
	29 Mar 1984	11:45	148766	25.61 (1225)	21.59 (1033)	1.41 / 1 (67)

		12:42	148923	25.30	22.62	0.85 / 16
				(1211)	(1082)	(41)
6	17 Dec 1983	16:45	9	28.40	28.48	0.31 / 1
				(1359)	(1363)	(15)
	29 Mar 1984	11:45	148740	26.32	21.74	1.06 / 1
				(1259)	(1040)	(51)
		14:00	148875	27.15	24.22	0.60 / 16
				(1299)	(1159)	(29)

A comparison of the results of the load tests performed after periods of undisturbed consolidation shows reasonable agreement among the several experiments.

It will also be noted that the increase in shear transfer capacity with time after driving occurs early in the consolidation process, being 0.83 ksf (40 kPa) after 500 minutes (Experiment 2), with additional increases to 0.98 ksf (47 kPa) after 4260 minutes (Experiment 1), and a further increase to only 1.06 ksf (51 kPa) after 103 days of consolidation (Experiment 6).

It is also of interest to note that, in a manner very similar to that discussed earlier for the experiments at the 178-ft (54.3 m) depth, both the initial static resistance and the cyclic minimum resistance exhibit additional increases beyond those associated with consolidation alone when time is allowed for reconsolidation and recovery of shear transfer capacity after cyclic degradation has been imposed on the soil, either early (Experiment 2) or late (Experiment 5) in the consolidation process.

Reasonable agreement is seen in the values of cyclic minimum resistance measured after short periods of reconsolidation in Experiments 2 and 3.

During Experiment 5, an even greater increase in the cyclic minimum resistance was observed, although the initial value on the first cyclic test, 0.70 ksf (34 kPa), is also larger than was recorded for the other experiments. The proportional increase, however, 34 percent, is almost the same as the increase of 37 percent in Experiments 2 and 3.

Although there is some variation among the initial values of excess total and pore pressures which were recorded at the end of driving, it should be kept in mind that the probes were all driven open-ended; thus, exact repeatability should not be expected. The magnitudes of excess pressure which are developed during pile driving are extremely sensitive to variations in the wall thickness of the pile, to the rate at which the pile penetrates, and to the relative volumes of soil which are either displaced outward, or enter the cutting shoe. Since it is impossible to obtain exact repetition in each of the factors affecting the magnitudes of the pressures which are created during the process, a reasonable amount of scatter is to be expected.

Much closer agreement was obtained for the values of pore pressure which were recorded after long periods of consolidation, with approximately 2 percent variation in the values recorded at the end of 72 hours of consolidation (Experiments 1, 2, and 5), and only 0.5 percent for those experiments in which longer periods of consolidation were allowed (Experiments 2, 5, and 6).

Similar agreement may be noted for the values of total pressure which are given for those experiments after long periods of consolidation (Experiments 2, 5, and 6), with a variation of 1.5 percent from the mean, or average, value.

6. SUMMARY OF RESULTS

In this chapter, the results of the experiments performed in each of the three soil strata will be summarized.

In the summary of results of the experiments at all depths except 208 feet (63.4 m) below the mudline, where the results of the experiments reported herein were used to establish the in situ pore pressures, the values of ambient pore pressure used to estimate the degrees of consolidation were based on measurements made along the 30-inch (76.2 cm)-diameter pile in April, 1985.

In the presentation of the results of the experiments at each depth, the consolidation data have been normalized, in order to eliminate the effects of variations in the magnitudes of initial excess pore pressure in the comparisons. The consolidation data were normalized by dividing the difference between the initial maximum excess pore pressure and the instantaneous values by the difference between the initial maximum excess pore pressure and the ambient pore pressure, then expressing the result as a percentage (i.e., multiplying by 100).

The shear transfer-displacement curves shown in this chapter are taken from those presented in Chapter 5; the curves have merely been superimposed for purposes of comparison. The relationships which are given for the shear transfer, the degree of consolidation, and the radial effective pressure are taken from the values tabulated in sections 5.2.4, 5.3.6, 5.4.5, and 5.5.7.

6.1 Summary of Results in Stratum I

Three experiments were performed in this stratum, all at a depth of 58-ft (17.7 m) below the mudline. As discussed in Chapter 5, the rate of consolidation was possibly affected by the method used to install the pile segment models; however, it is not felt that the magnitudes of shear transfer were affected appreciably, if the degree of consolidation is properly taken into account.

The dissipation of excess pore pressures with time after installation of the pile segment models at this depth are shown in Plate 6.1. The results of only two of the three experiments are shown; as discussed in Chapter 5, the consolidation in Experiment 1 at this depth was adversely affected by the disturbance caused by drilling in an adjacent boring.

As seen in the plate, excellent agreement was observed between the consolidation behavior observed during the two experiments. Although a lack of agreement would not be unexpected, due to the uncertainty in the initial conditions, it can be seen that excellent repetition was achieved, indicating that consolidation occurred under very similar drainage conditions.

It had been expected that the consolidation would be nearly complete in 72 hours, based on the results of some earlier onshore experiments using the probes in a similar clay. As seen in Plate 6.1, the rate of consolidation which had been predicted for this layer of clay had been badly overestimated.

It should be realized, however, that at the time the experiments were performed, little was known about the process of consolidation around driven piles; the effects of plugging were thought to be only an increase in the magnitudes of excess lateral pressures which were developed; the increase in the time required for consolidation had not been considered. In addition, the ambient pore pressures which existed in the soil were not known.

The shear transfer-displacement behavior which was recorded after periods of undisturbed consolidation during Experiments 1 and 2 are given in Plate 6.2. Again, excellent agreement is shown; the difference of 0.02 ksf (1 kPa) in the magnitudes of shear transfer is negligible.

A comparison of the shapes of the curves indicates somewhat softer, more plastic response of the soil during Experiment 1 (the lower curve), during which the consolidation was allowed to proceed uninterrupted for 72 hours prior to the test. This suggests that plastic deformations are occurring in a thicker layer of clay, with the shear deformation extending a further radial distance from the pile wall.

The similar magnitudes of peak shear transfer after consolidation times of 8 and 72 hours indicates that the shear transfer capacity of the soil was developed very soon after driving; however, in none of the experiments was the degree of consolidation greater than 50 percent, thus, no conclusions regarding the long-term behavior can be drawn.

The shear transfer-displacement behavior which was recorded at the end of similar periods of reconsolidation following an initial series of cyclic tests during Experiments 2 and 3 are given in Plate 6.3. As seen in the plate, similar values of peak shear transfer were recorded; however, the soil exhibited a greater stiffness in the experiment in which a lower value of shear transfer was observed. This suggests that, for the upper curve, the shear deformations extended a farther radial distance from the pile wall than for the lower curve. Thus, the higher value of shear transfer may result from an equal (or lower) value of shear stress in the soil.

In Plate 6.4a, the relationship among the values of peak shear transfer recorded in each load test and the degree of consolidation which had been reached at the time of the test is shown. As expected, generally good agreement can be seen. Although the values of shear transfer during the retests may have been affected somewhat by the intermediate tests, it can be seen that the effects were not great. Again, the higher value of shear transfer for the retest in Experiment 3 may be partly due to failure occurring at a larger radial distance from the pile wall.

Also shown in Plate 6.4a is the relationship among the values of cyclic minimum shear transfer and the degree of consolidation. Except for Experiment 3, in which the value falls among the values of peak shear transfer, there seems to be no increase in the magnitude of the degraded resistance with increases in the degree of consolidation. Again, it should be stressed that the degree of consolidation is very low; clearer trends might have emerged had additional consolidation been allowed.

In Plate 6.4b, the values of maximum and cyclic minimum shear transfer shown in Plate 6.4a are plotted against the values of radial effective pressure which are based on the values of soil pressure measured immediately prior to performing the cyclic tests. As seen in the figure, no clear trends are evident, except for those experiments in which a load test was performed after a period of undisturbed consolidation.

It should be noted that, if a direct relationship exists between the peak or the cyclic minimum shear transfer and the radial effective consolidation pressure, the points in Plate 6.4b should fall on one or two failure envelopes (one for the peak and one for the cyclic minimum shear transfer), which may or may not have a non-zero shear intercept at zero radial effective pressure. As seen in the plate, no such trends are evident.

6.2 Summary of Results in Stratum II

Five experiments were performed in this soil stratum; four with the 3-inch (7.62 cm)-diameter probes and one with the full-displacement, 1.72-inch (4.37 cm)-diameter pile segment model. Of the four experiments with the 3-inch (7.62 cm)-diameter probes, one was performed with the cutting shoe being allowed to plug prematurely, and three were performed with thin-wall, unplugged cutting shoes. All the experiments were performed at a depth of 148-ft (45.1 m) below the mudline.

The progress of consolidation during the experiments performed with the models having thin-wall, unplugged, cutting shoes are shown in Plate 6.5. Included in the plate are the results of Experiments 1 and 3. The results of Experiment 4, during which the soil pressures increased with time during the first four hours after installation, are not included.

As seen in the plate, the pore pressures also increased with time shortly after installation during the two experiments shown, but reached maximum values within ten minutes after installation, rather than the period of four hours in Experiment 4.

Since similar behavior was observed at the end of some of the load tests, the increase in pore pressure at the end of driving may be related to dilation of the adjacent clay during plastic slip as the pile was driven into the soil.

Again, excellent agreement is shown for the time-rate of dissipation of excess pore pressure during the two experiments. Except for the latter part of consolidation in Experiment 1, which may have been affected by drilling in adjacent borings, the two curves are virtually identical.

It is also of interest to note that, following cyclic load tests, the pressures tend to return to the same path, which suggests that the disturbances created in the soil by cyclic loading are confined to a small circumferential layer of clay very near the pile, and that the lateral pressures in the soil mass are not appreciably affected.

The dissipation of excess pore pressure with time during Experiments 1, 2, and 5 are given in Plate 6.6. As seen in the plate, the time-rate of dissipation of excess pressure during Experiment 2, in which the cutting shoe was intentionally allowed to plug prematurely, is slightly faster than that recorded during Experiment 1, in which the cutting shoe did not plug until after the tip was well below the final depth of the pressure transducers.

Such behavior was unexpected; the time-rate of dissipation of excess pore pressure would be expected to be slower for Experiment 2 because of the increased radial extent of disturbed and remolded soil. The similarity in behavior exhibited by the probes suggests that either the open-end shoes also became plugged, or that the major effects of premature plugging on the soil occurred below the depth of the pressure transducers. Although the radial pressures were larger for the prematurely-plugged model, the radial path of flow of pore water must have been very nearly the same as for the unplugged pile models, else the rate of dissipation would have been slower.

As seen in the plate, the excess pore pressures dissipated more rapidly during the experiment with the 1.72-inch (4.37 cm)-diameter probe; the behavior indicates the first series of cyclic tests, during which time the shear transfer increased by 50 percent. The same relative increase was observed in Experiment 4, in which 39 hours was allowed for reconsolidation, which suggests that the recovery of shear transfer capacity after cyclic degradation occurs rapidly upon the cessation of loading.

Plate 6.9 contains the curves of shear transfer versus displacement which were recorded on the last cycle of the load tests performed during Experiments 1, 2, and 3. As noted earlier, little cyclic degradation was evident in Experiments 4 and 5; therefore, the curves recorded during these experiments are not shown. The highest value of cyclic minimum shear transfer, obtained during Experiment 2, may have been affected somewhat by load rate, since the test was performed during a period in which wave loading of the structure was affecting the tests. Except for this curve, excellent agreement can be seen among the results of the tests, with the magnitude of the cyclic minimum shear transfer generally increasing with the increase in the degree of consolidation.

The development of shear transfer with the increase in the degree of consolidation is shown in Plate 6.10a. As seen in the figure, three different patterns of behavior emerged.

During Experiments 4 and 5 (the topmost line), the maximum shear transfer increased linearly with the increase in the degree of consolidation, at a level of shear transfer which was higher than that observed during the first three experiments. During Experiments 4 and 5, little cyclic degradation was observed.

For Experiments 1, 2, and 3, the maximum shear transfer (the middle line) also increases linearly with the degree of consolidation, but at a lower level, and at a slower rate than during Experiments 4 and 5. As seen in the values of shear transfer along the bottom line, the degree of consolidation has little effect on the cyclic minimum shear transfer.

In Plate 6.10b, the values of shear transfer shown in Plate 6.10a are plotted against the values of radial effective consolidation pressure which were based on static total and pore pressures measured prior to the tests. As seen in the plate, direct relationships among the values of shear transfer and the radial effective pressure were not observed. If a direct relationship existed, then the points in the figure should have fallen on one or two failure envelopes. Again, no such trend is evident.

6.3 Summary of Results in Stratum III

The major emphasis of the research program was placed on performing experiments in the soil in the deepest stratum, which lay below a depth of 160-ft (48.8 m) below the mudline. This soil was felt to be of primary importance for two reasons: 1) the major part of the axial pile capacity during the large-diameter instrumented pile tests would be developed in this stratum and, 2) the clay in this stratum was felt to be more representative of the highly-plastic clays found at the deep-water Green Canyon site.

Ten experiments were performed in this stratum; four at a depth of 178-ft (54.3 m) below the mudline, and six at a depth of 208-ft (63.4 m) below the mudline. Nine of the experiments were performed using the 3-inch (7.62 cm)-diameter pile segment models with thin-wall cutting shoes; one was performed using the 1.72-inch (4.37 cm)-diameter X-probe.

Plate 6.11 contains the records of the dissipation of excess pore water pressure from the experiments performed at the 178-ft (54.3 m) depth.

As seen in Plate 6.11, the rate of dissipation of excess pore pressure recorded for the full-displacement X-probe was the same as that recorded for the 3-inch (7.62 cm)-diameter, partial-displacement probes. This behavior is in direct conflict with that shown in Plate 6.6, in which the time-rate of dissipation of excess pore pressure was faster for the smaller-diameter probe. The behavior indicates, however, that the rate of dissipation of excess pore pressure is not a function of pile diameter alone, but also of the relative wall thickness (as reflected by the degree of cavity expansion during installation).

The shear transfer-displacement behavior which was recorded after periods of undisturbed consolidation during the experiments with the 3-inch (7.62 cm)-diameter probes are shown in Plate 6.12. Included in the figure is the initial loading to failure during the load-controlled, one-way cyclic tests in the first experiment.

The differences in the peak shear transfer among the three experiments is not large; however, the largest value, 0.66 ksf (31.6 kPa), was recorded during the experiment in which the period of undisturbed consolidation was almost 72 hours, with only 8 hours being allowed in the remaining two experiments.

A comparison of the values of shear transfer during these tests with those recorded immediately after driving, however, again indicates that the development of shear transfer capacity occurred very early in the process of consolidation, with setup factors of three to nine being observed in the first 8 hours of consolidation, and with only minor increases thereafter. It was during these experiments, however, that the first indication of long-term increases in resistance for periods of consolidation longer than 8 hours were observed.

The curves of cyclic minimum shear transfer versus displacement which were recorded during the last cycle of each of the load tests performed with the 3-inch (7.62 cm)-diameter probes are shown in Plate 6.13. As shown in the Plate, excellent agreement is shown among the values of post-peak resistance recorded during three of the experiments, with variations occurring only in the peak shear transfer. The highest curve was obtained during the experiment in which 72 hours was allowed for consolidation; the remaining curves were obtained at consolidation times ranging from 8 to 20 hours.

The values of maximum and cyclic minimum shear transfer recorded at varying degrees of consolidation during the experiments are given in Plate 6.14a. As seen in the plate, the behavior again falls into three distinct patterns; 1) the highest values of shear transfer were observed during retests, 2) the initial peak shear transfer during tests after undisturbed consolidation were somewhat lower, and 3) the cyclic minimum shear transfer from all tests was still smaller. As seen in the figure, the values of shear transfer in each regime increased with an increase in the degree of consolidation.

The values of shear transfer shown in Plate 6.14a are also shown in 6.14b, plotted against the value of radial effective pressure that existed just prior to performing the load tests. The values of minimum resistance shown at a value of radial effective pressure of 4.72 ksf (226.0 kPa) were obtained using the 1.72-inch (4.37 cm)-diameter X-probe, in which no cyclic degradation was observed. The values shown in the Plate are the values of the reduced post-peak residual resistance.

As seen in Plate 6.14b, no direct relationship was obtained between the values of initial peak shear transfer and radial effective pressure. A trend in the cyclic minimum resistance is evident, however.

The normalized consolidation behavior observed during the experiments at the 208-ft (63.4 m) depth are shown in Plate 6.15.

During these experiments, two distinct patterns of behavior are evident. For those experiments during which the pore pressures increased for the first few minutes after installation, the consolidation process took a longer time. For those experiments in which the excess pore pressures immediately began to dissipate, consolidation was more rapid. The reasons for such behavior are not understood; however, the tendency for the pore pressures to increase in the early part of consolidation may be related to dilation of the near-pile clay during driving, with similar behavior being noted after the conclusion of some of the cyclic load tests.

The results of the first cycle of loading during the cyclic tests performed after varying periods of undisturbed consolidation during Experiments 1, 2, and 6 at the 208-ft (63.4 m) depth are shown in Plate 6.16. The results of these tests clearly show the increase in shear transfer capacity with time, with a value of 0.83 ksf (39.7 kPa) after 8 hours of consolidation, 0.98 ksf (46.9 kPa) after 72 hours, and 1.06 ksf (50.8 kPa) after 105 days. It should be noted that the increases which were observed were restricted to increases in the peak resistance on the first cycle of loading; the cyclic minimum resistance in the three experiments was 0.60, 0.63, and 0.60 ksf (28.7, 30.2, and 28.7 kPa), respectively, or, for all practical purposes, equal values.

The behavior observed during the three experiments agrees very well with that which had been expected; i.e., a very rapid initial increase in shear transfer, which then continued to slowly increase with time. The agreement among the values of cyclic minimum resistance in the three experiments also suggests that the shear surface which was formed was nearly identical in the three experiments, in terms of both radial distance from the pile wall and in the void ratio of the clay which was sheared.

The shear transfer-displacement behavior which was recorded at the end of periods of reconsolidation after cyclic degradation are shown in Plate 6.17. The curves shown in the plate were taken from the first cycle of loading in the second series of cyclic tests in Experiments 2, 3, and 5 at the 208-ft (63.4 m) depth.

A comparison of Plates 6.16 and 6.17 indicates that, with one exception, the resistance measured during the retests was higher than that recorded during equal periods of consolidation in experiments where an intermediate cyclic test was not performed.

Based on the examinations of the layers of clay which adhered to the probes after removal of the probes from the borings, and on the fact that the shear transfer capacity measured during the retests was greater than the values measured during experiments with equal periods of undisturbed consolidation, the increases in resistance during reconsolidation can be identified as arising from two sources: 1) an increase in shear strength in the clay accompanying a decrease in void ratio as the shear-generated excess pore pressures dissipate, and, 2) an outward shift of the shear zone away from the initial failure surface as the clay near the initial slip surface increases in shear strength during reconsolidation.

For a pile model having a diameter of only three inches, a small increase in the radial distance to the soil-soil failure surface will increase the calculated value of shear transfer significantly. A comparison of the cyclic minimum resistances measured during the retests of the probes show increases ranging from 17 to 37 percent, as compared with values measured during the load tests which were performed after undisturbed consolidation. Although the increases in the cyclic minimum shear transfer cannot be attributed solely to the change in geometry of the shear surface, an increase in the range of 10 to 15 percent is likely, based on the observations made of the multiple failure surfaces in the soil which adhered to the probes during removal.

For prototype-sized piles, increases of such magnitude may not occur. If the increase in the radial distance to the shear zone is related to the thickness of the layer of severely-distorted and disturbed soil, then an equivalent linear increase in the diameter of the cylindrical shear surface would have little effect on the capacity of the pile. If, however, the increase in the radial distance is a function of pile diameter, then increases in resistance similar to those shown in Plate 6.17 would also be expected.

The curves of shear transfer versus displacement which were recorded at the end of the cyclic load tests performed during the experiments at this depth are shown in Plate 6.18. The four lowest curves were obtained during cyclic tests which were performed at the end of undisturbed consolidation, with the consolidation times ranging from 8 hours to 105 days. It can be seen that excellent agreement is shown, especially for the reduced post-peak resistance at large displacements.

The three middle curves were taken from experiments in which periods of reconsolidation after initial cyclic degradation ranging from 6 to 72 hours were allowed prior to retesting the probes.

The top curve was recorded during Experiment 5, in which 101 days were allowed for reconsolidation and recovery after cyclic degradation prior to retesting the probe.

A comparison of the curves indicates that the increase in the cyclic minimum resistance with time was dramatic, and was approximately of the same order of magnitude as the increase in the initial peak resistance, as seen in a comparison of Plates 6.16 and 6.17, indicating similar degrees of loss of resistance during each cyclic test.

In Plate 6.19a, the increases in shear transfer with increases in the degree of consolidation are clearly shown. For those experiments in which the tests were performed at the end of undisturbed consolidation (with the exception of Experiment 4), the increase in shear transfer is a linear function of the degree of consolidation. Little effect of the increase in the degree of consolidation can be seen for the cyclic minimum resistance which was recorded during these tests, however.

Again, with the exception of one experiment, the values of initial peak and cyclic minimum shear transfer recorded during load tests performed after periods of reconsolidation following cyclic degradation were consistently higher than those measured during tests after undisturbed consolidation, at comparable degrees of consolidation.

The values of shear transfer shown in Plate 6.19a are again shown in Plate 6.19b, plotted against the value of radial effective pressure that existed prior to each load test. Excellent agreement is shown for the initial peak shear transfer recorded in load tests which were performed at the end of various periods of undisturbed consolidation, with the points falling on a straight line having a non-zero shear transfer intercept.

No such relationship can be established for either the cyclic minimum shear transfer or for the peak shear transfer which was measured at the end of periods of reconsolidation and recovery from cyclic degradation.

7.0 SUMMARY AND CONCLUSIONS

In this chapter, the results of the experiments performed with the small-diameter pile segment models will be evaluated, and the applicability of the results to the design of prototype-size foundation piles will be discussed.

7.1 Evaluation of the Experimental Results

The pile segment models which were developed for the in situ determination of the axial shear transfer capacity of soft soils have been demonstrated to have served the intended purpose very well. In only two of the eighteen experiments did a failure of the instruments prevent the successful completion of the experiments which had been planned. In one instance, the performance of a duplicate experiment in a parallel boring served to replace the information which had been lost. For the long-term test, in which the consolidation was to have been extended for three months, a duplication of the experiment was not feasible.

During the major part of the work, the instruments were extremely stable and drift-free. Only in the experiments in which the models were in place for a period of three months did the drift create uncertainty in the measurements, and then only in the total pressure measurements. In none of the experiments did evidence of drift arise with the axial load cells, which were of primary importance; nor did problems arise with the pore pressure transducers.

Excellent agreement was noted in the consolidation behavior among the various experiments at each depth in Strata I and II, and for two depths in Stratum III. In Stratum III, two modes of consolidation behavior were observed, the reasons for which are unclear.

Exactly comparable experiments could not be performed with both probe sizes in the fully-plugged end condition mode, thus a comparison of the effects of diameter alone on the rate of consolidation could not be made.

Very good agreement was observed among the values of shear transfer which were measured during the experiments at each depth, especially among the load tests which were performed after various periods of undisturbed consolidation.

In all the experiments, the peak shear transfer which was recorded increased with the degree of consolidation. Except for the 178-ft (54.3 m) depth, the magnitudes of cyclic minimum shear transfer were shown to be unrelated to the degree of consolidation.

For those experiments in which load tests were performed after periods of reconsolidation, during which the soil recovered shear transfer capacity after cyclic degradation, very clear patterns of behavior were also shown. In each of the retests, both the peak and the cyclic minimum shear transfer capacity were observed to equal or exceed the values which were recorded after an equivalent period of undisturbed consolidation.

With the exception of the peak shear transfer recorded at the end of periods of undisturbed consolidation, no consistent relationships could be demonstrated among the values of shear transfer and the corresponding values of radial effective pressure. Since both the shear transfer and the radial effective pressure increased during monotonic consolidation, consistent relationships should have been expected. The parallel development of increased shear transfer capacity and radial effective pressure does not imply that the shear transfer is a function of the radial effective pressure, but merely that the increases in both are natural consequences of the process of consolidation.

An examination of the variations in the radial pressures during the cyclic load tests shows that significant changes in the pore pressure accompany plastic slip in the soil. In attempting to interpret the significance of the variations in the pore pressures, it should be kept in mind that the pore pressures which were measured were those existing at the pile wall, and not on the soil-soil failure surface. Since, during plastic slip, steep radial gradients in the pore pressure in the near-pile clay undoubtedly exist, the pore pressures measured at the pile wall merely indicate the direction of the changes in pore pressure on the failure surface, and not the actual magnitudes of pressure. Thus, the values of radial effective pressure based on measurements recorded during active loading are of very limited utility, since the values are based on differences between values of total pressure which are almost correct, and values of pore pressure which are not. Therefore, the instantaneous values of radial effective pressure cannot be used to establish the actual stress paths followed by the soil during shear. The pore pressure measurements merely indicate the patterns of volume change which are occurring in the clay near the failure surface during cyclic loading.

The relationships between the shear transfer and the instantaneous values of soil pressure which are measured at the pile wall are thus not directly comparable with the results of triaxial compression tests. In triaxial compression tests, the shear stress is calculated as one-half the difference between the applied (and controlled) normal stresses; a direct relationship between the shear stress and the major principal normal stress is thereby ensured. During axial pile loading, a shear stress is directly applied to the clay adjacent to the pile wall, the magnitude of which is completely independent of the normal stresses. No relationship between the magnitude of the applied shear stress and the radial stresses can be inferred. In triaxial compression tests, the dependence of the shear stress on the normal stress is enforced, and the relationships between the normal stresses and the calculated shear stress (the stress path) may be interpreted on the basis of past experience in such tests. During axial pile loading, the shear stress in completely independent of the normal stress, and the normal stresses are uncontrolled (and, to some extent, unmeasurable, due to the radial gradient in pore pressure). Thus the relationships between the shear transfer and the radial effective pressure may not be subject to the same interpretation.

Since pure shear does not directly create volumetric strain, only minor changes should be observed in the normal stress during axial pile loading. Any significant change in the normal radial pressures must therefore be interpreted in terms of volume changes in the near-pile clay. Unlike triaxial compression tests, in which the changes in the calculated shear stress are directly caused by changes in the applied normal stress, the increases and decreases in the radial pressures during plastic slip must be due to expansions and contractions of the clay confined within the surface of slip. During pile load testing, the changes in radial pressure are responses to, and not sources of, changes in shear stress and strain.

Once plastic slip has occurred on a circumferential surface, the dominant effects of cyclic loading will be confined to the clay contained within the boundaries of the surface. The outside boundary of the failure surface will, essentially, act as a semi-permeable membrane, transferring shear stresses to the adjacent soil and allowing flow into, and away from, the clay contained within the surface. Due to the considerable differences in 1) inter-particle bonding; 2) particle orientation; and 3) shearing resistance, the clay outside the surface will not experience the severe effects of reversals of plastic strain, slip, and volume changes that are being created in the clay within the shear zone.

The patterns of variation in the pore pressure during and after the cyclic tests suggest that cyclic degradation in shear transfer capacity in a clay soil is at least partially due to radial flow of pore water into a thin layer of clay near the slip surface, with the changes in shearing resistance in the clay being associated with increases in void ratio and moisture content. It is by no means suggested that changes in moisture content alone are responsible for the decreases in resistance, but are one component of the mechanism.

The recovery in shear transfer capacity after cyclic degradation is also at least partially due to the expulsion of pore water from the layer of clay between the slip surface and the pile wall, as the excess pore pressures dissipate. The dissipation of these excess pore pressures results in an additional increment of consolidation and decrease in void ratio which is greater than that which would result from monotonic consolidation alone, in the absence of cyclic loading. In each experiment in which sufficient time was allowed for recovery in resistance, both the maximum and the cyclic minimum shear transfer increased to values greater than those measured after comparable periods of undisturbed consolidation.

Of all the fundamental strength properties of clay soils, the resistance to shear along a slip surface after many reversals of failure is the only property which is solely a function of the mineralogical content and the void ratio of the clay, with any effects of the stress history or depositional environment being destroyed by the repeated imposition of reversed plastic strain. Thus, in evaluating the effects of consolidation (i.e., reductions in void ratio) on the shear transfer capacity of the soil, the magnitude of the cyclic minimum resistance should be the most reliable standard.

During the cyclic load tests performed at the end of periods of undisturbed consolidation in the experiments in Stratum I and III, it was noted that the cyclic minimum shear transfer was relatively unaffected by the degree of consolidation, although the values of peak initial shear transfer were shown to increase.

Such behavior suggests that a portion of the initial maximum resistance of the soil may be due to the formation of physico-chemical bonding between the clay platelets, which results in additional increases in shearing resistance beyond those which are due to the decreases in void ratio alone. Upon the application of reversals of plastic slip, the inter-particle bonds are broken, and the shearing resistance reduces to a consistent minimum value, which is dependent solely on the void ratio. This behavior, defined as thixotropy, has long been associated with time-dependent increases in the shear strength of remolded samples of highly plastic clays under conditions of constant moisture content and void ratio.

When the shear-generated excess pore pressures were allowed to dissipate after the conclusion of the cyclic tests, the values of cyclic minimum resistance increased to values which ranged from 17 to 37 percent greater than the values which were observed in tests at comparable degrees of undisturbed consolidation. Thus, the void ratio of the clay must have decreased, with a simultaneous increase in the shearing resistance. Although the magnitudes of the measured values of the cyclic minimum shear transfer were possibly increased somewhat by an outward shift of the slip surface, the dominant effect was that of an increase in the shearing resistance along the slip surface, which, again, is solely a function of the void ratio.

7.2 Conclusions and Recommendations

In order to extrapolate the results of the experiments with the small-diameter tools to the axial load behavior of prototype-size piles, several aspects of the behavior must be considered.

The relationships among the diameter and the wall thickness on the rate of consolidation (and thereby the time-rate of development of shear transfer capacity) must be addressed. Because of the very limited number of experiments which were performed with the end conditions varied (i.e., the degree of end-plugging), the results of the experiments reported herein are felt to be insufficient to fully explore such relationships.

The patterns of behavior noted among the development of shear transfer capacity and the dissipation of excess pore pressure (and thereby the magnitudes of radial effective pressure) suggest that either total stress or effective stress approaches to the prediction of the ultimate, or long-term, static axial capacity of piles should be valid. It should be recognized, however, that, since a clay soil does not display truly frictional behavior, the end result of each method would be the same; a prediction of the magnitude of the undrained shear strength at the end of a period of monotonic consolidation.

In the development of axial pile design methods, it will be necessary to include consideration of the effects of the degree of end-plugging on the consolidation behavior, so that the time-rate of development of axial pile capacity may be properly estimated.

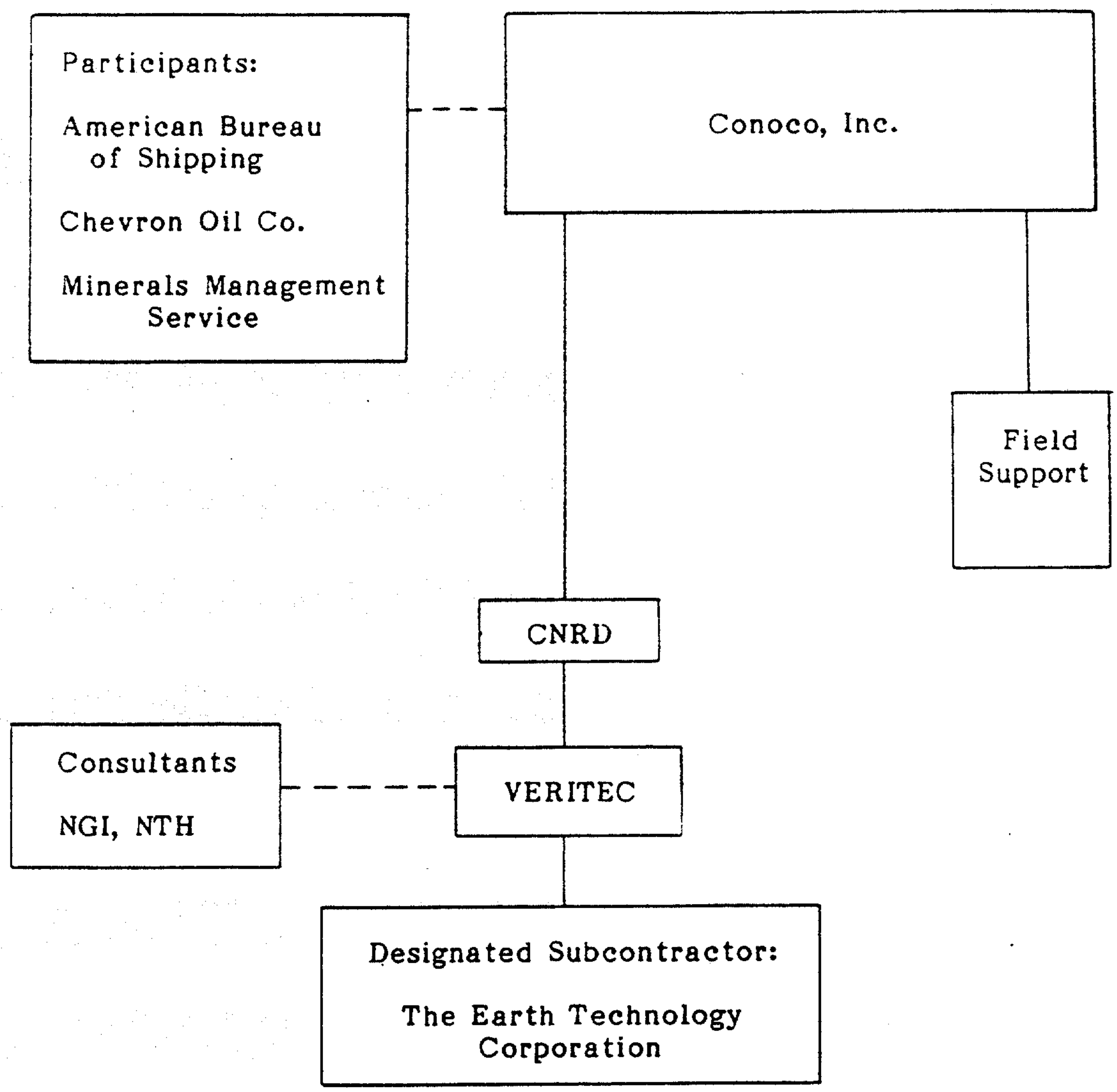
In the design and analysis of pile foundations which consist of large-diameter piles, the estimation of the ultimate, or long-term, axial capacity is not sufficient; such piles may not experience sufficient consolidation time to achieve the ultimate axial capacity during the life of the structure.

The lack of agreement among the values of pore pressure or radial effective pressure and the magnitudes of the cyclic minimum shear transfer or the values of shear transfer capacity which were observed following periods of recovery suggest that neither total nor effective stress approaches (as presently used) are directly applicable to the problems of cyclic degradation and the recovery in shear transfer capacity.

In the absence of clear relationships among the degree of consolidation, the radial effective pressure, the magnitudes of the cyclic minimum shear transfer, and the degree of recovery in the maximum shear transfer capacity, a mechanism for cyclic degradation was proposed, which was based solely on the observations of the directions of change in pore pressures during cyclic loading. The proposed mechanism includes consideration of the effects of the migration of pore water into, and away from, a thin layer of plastically-strained clay, but does not consider other factors, such as the effects of particle rearrangement (remolding) on the shearing resistance. Since such a mechanism does not lend itself to the development of quantifiable design procedures, the estimation of the magnitudes of cyclic degradation must, at present, remain empirical.

APPENDIX 1

ILLUSTRATIONS FOR CHAPTER 1

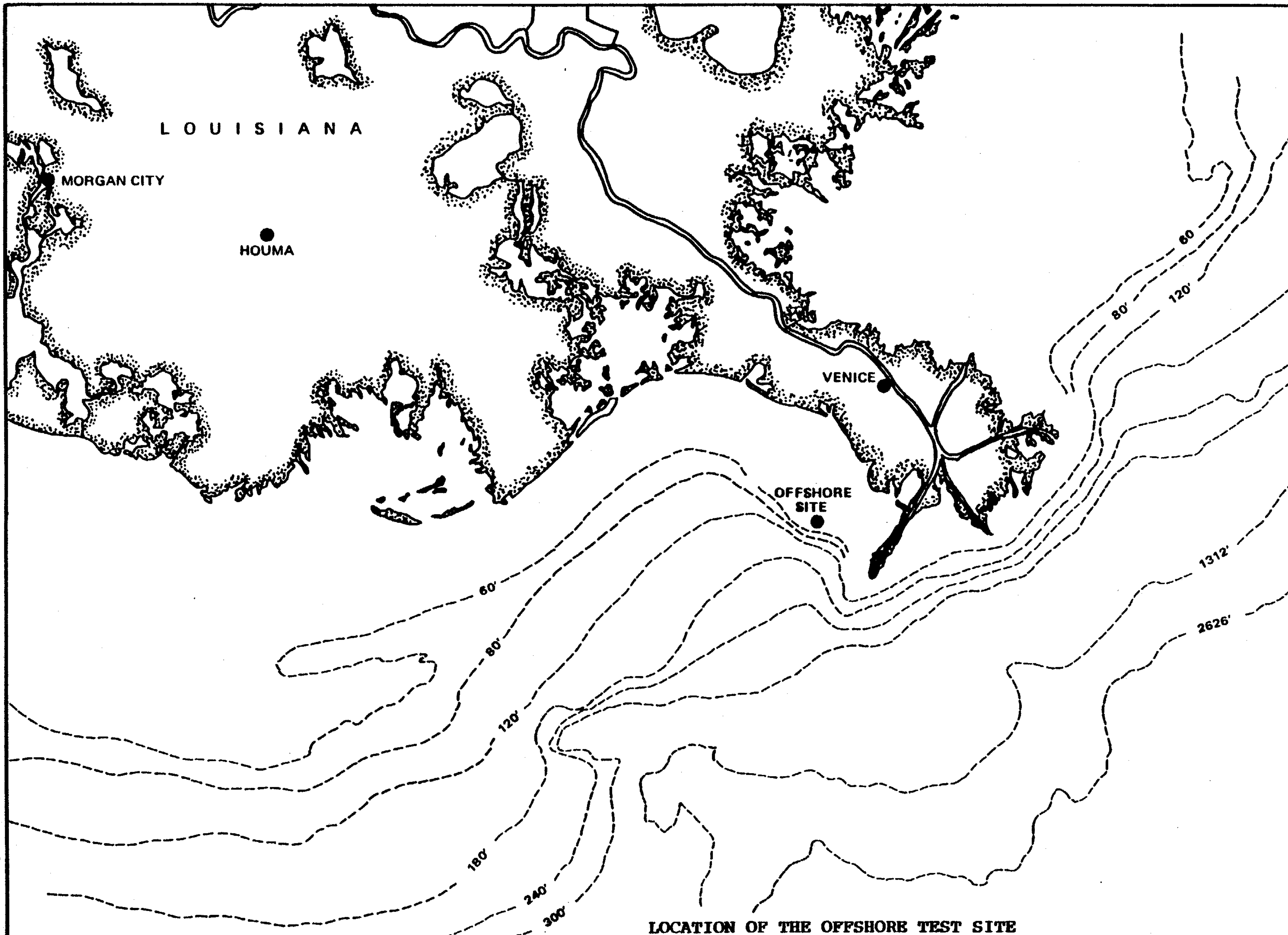


PROJECT ORGANIZATION CHART

Plate 1.1

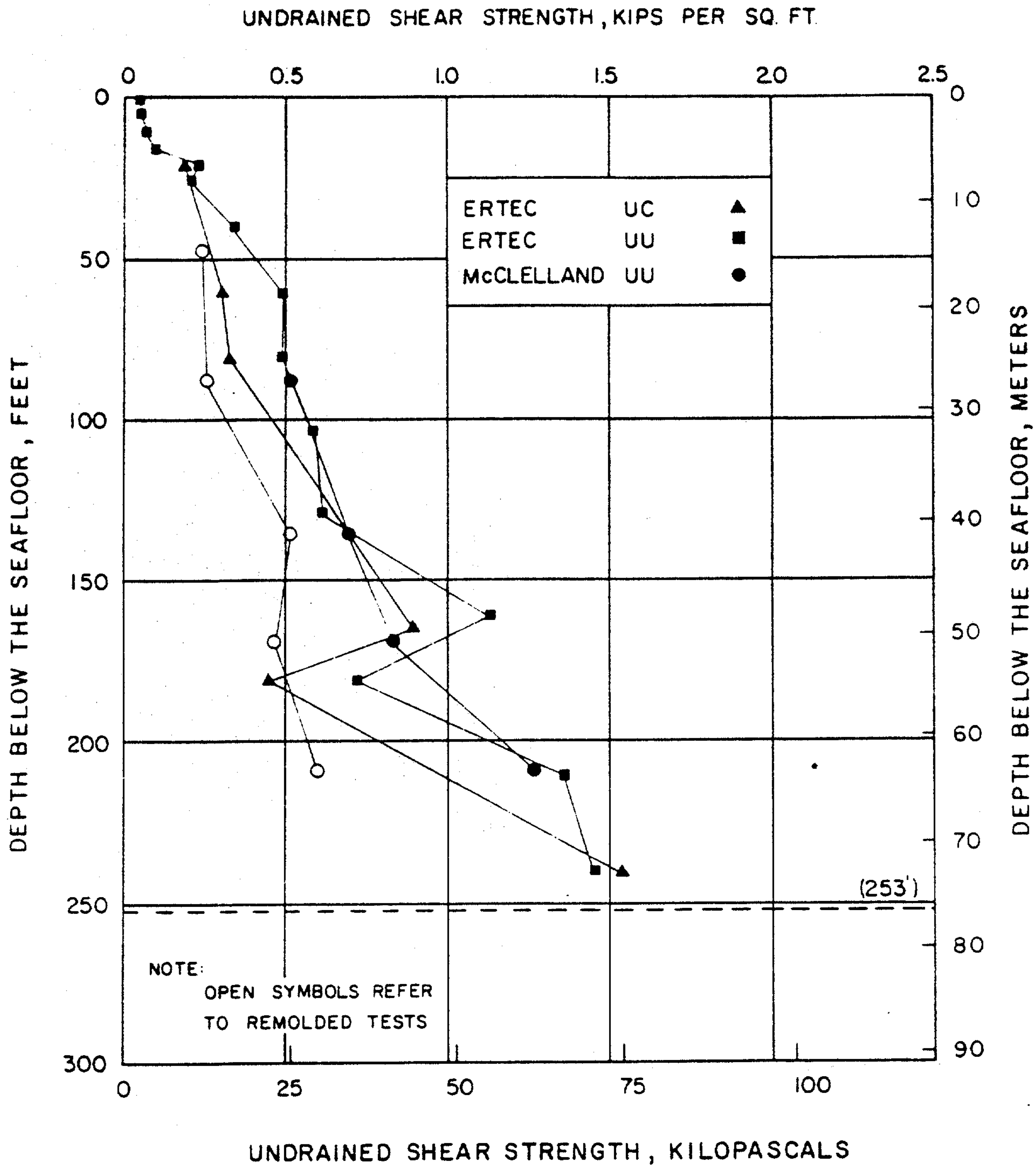
APPENDIX 2

ILLUSTRATIONS FOR CHAPTER 2

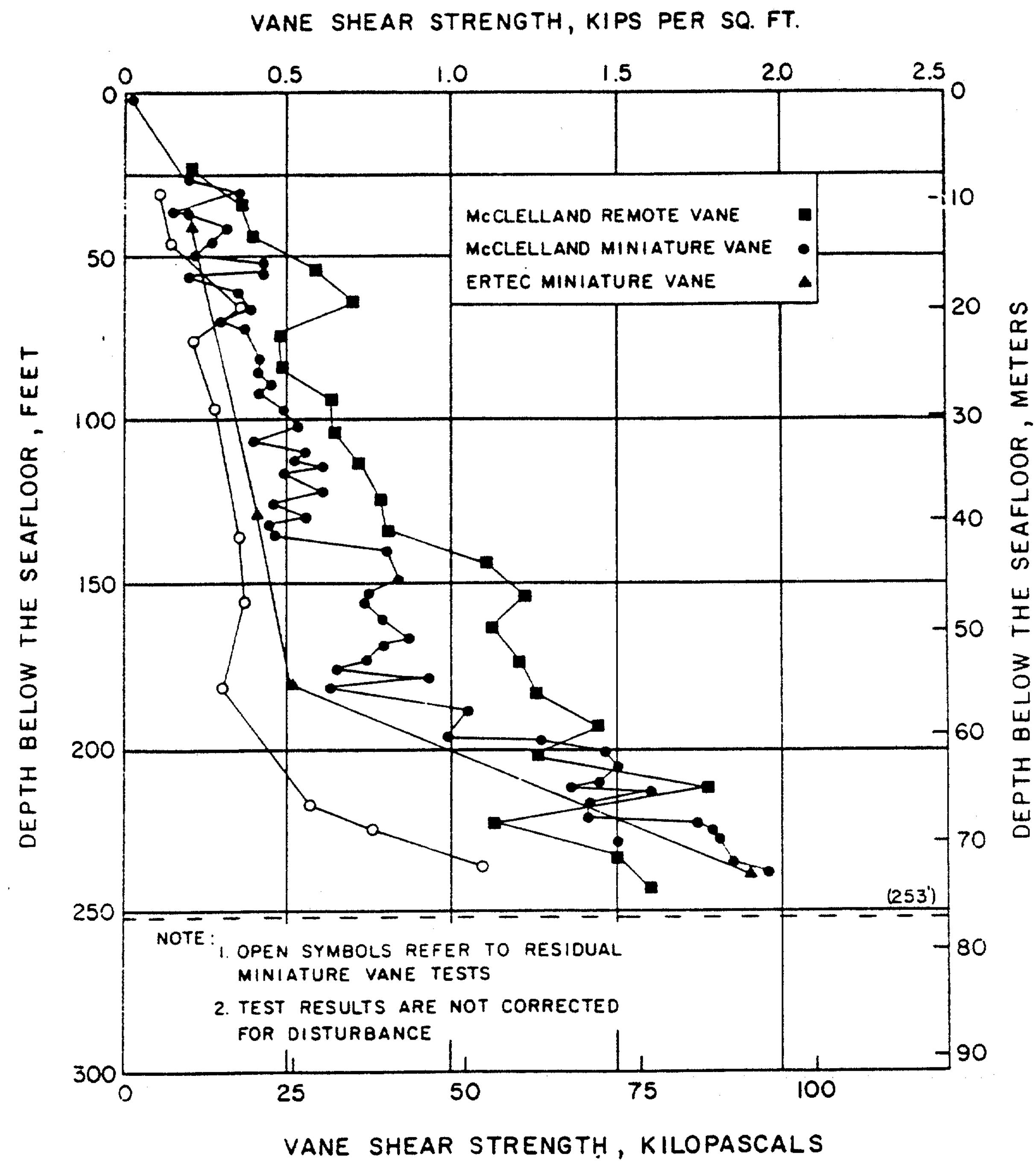


LOCATION OF THE OFFSHORE TEST SITE

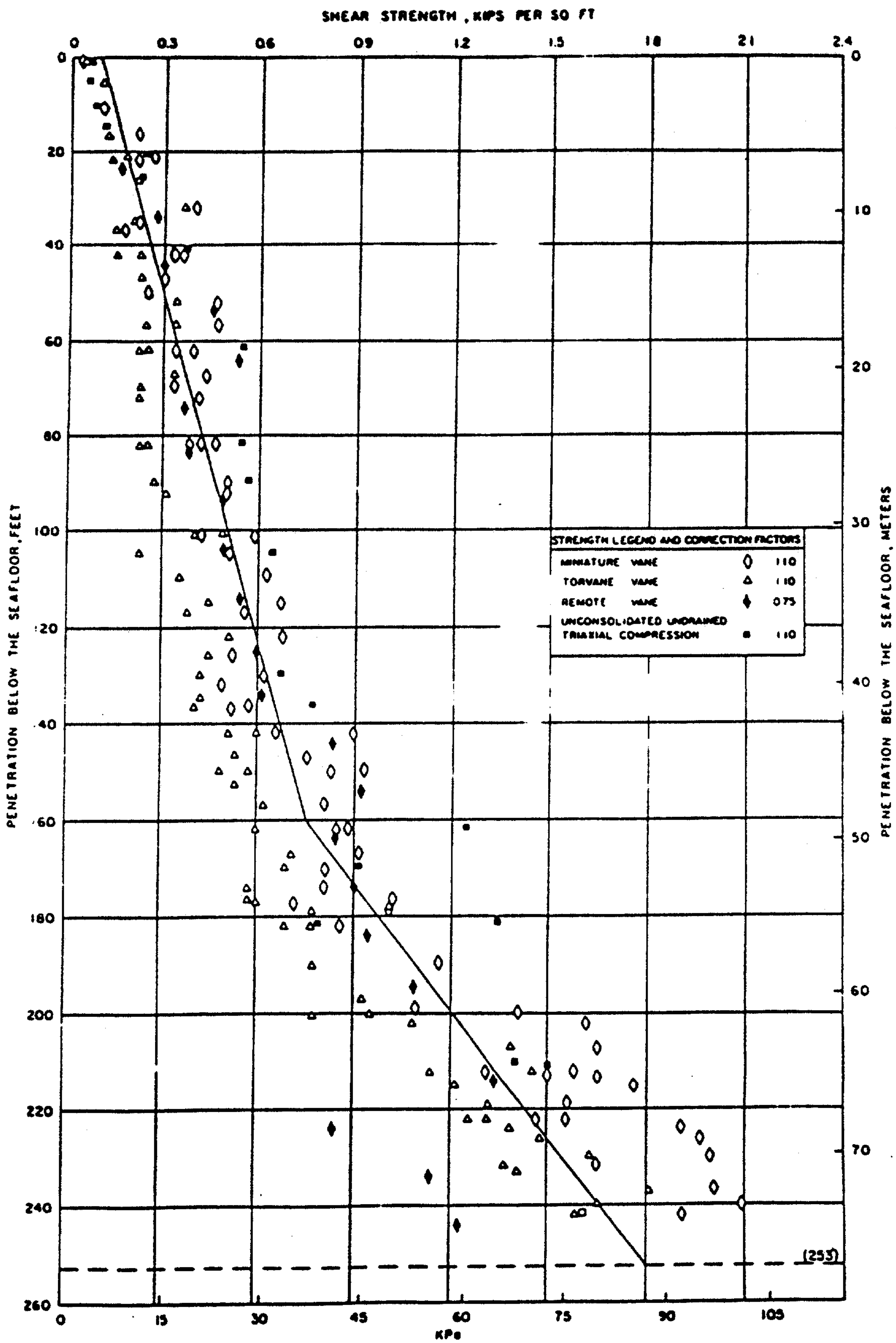
Plate 2.1



UNDRAINED SHEAR STRENGTH VERSUS PENETRATION
UU AND UC TESTS

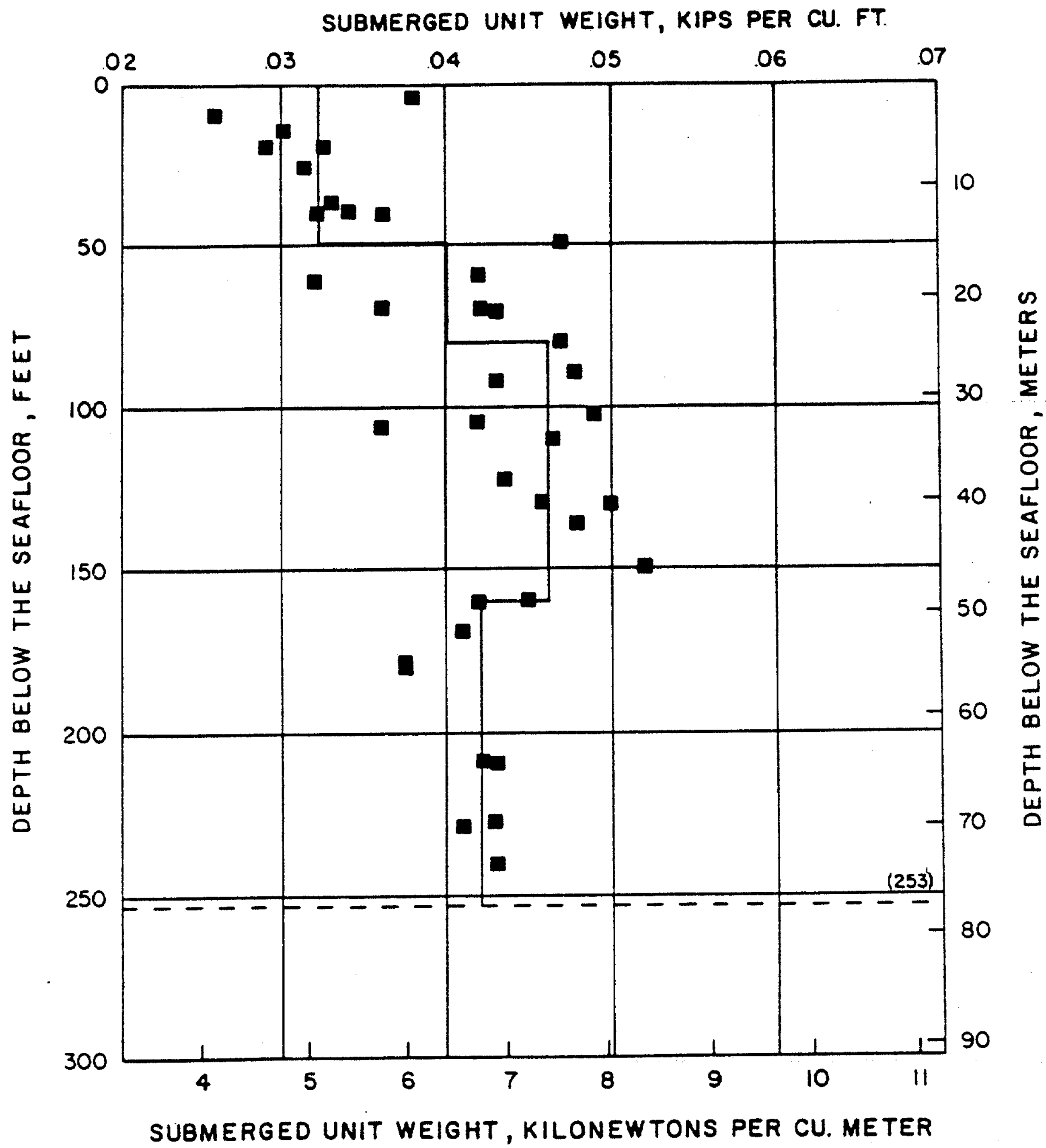


MINIATURE VANE AND REMOTE VANE SHEAR STRENGTH VERSUS PENETRATION

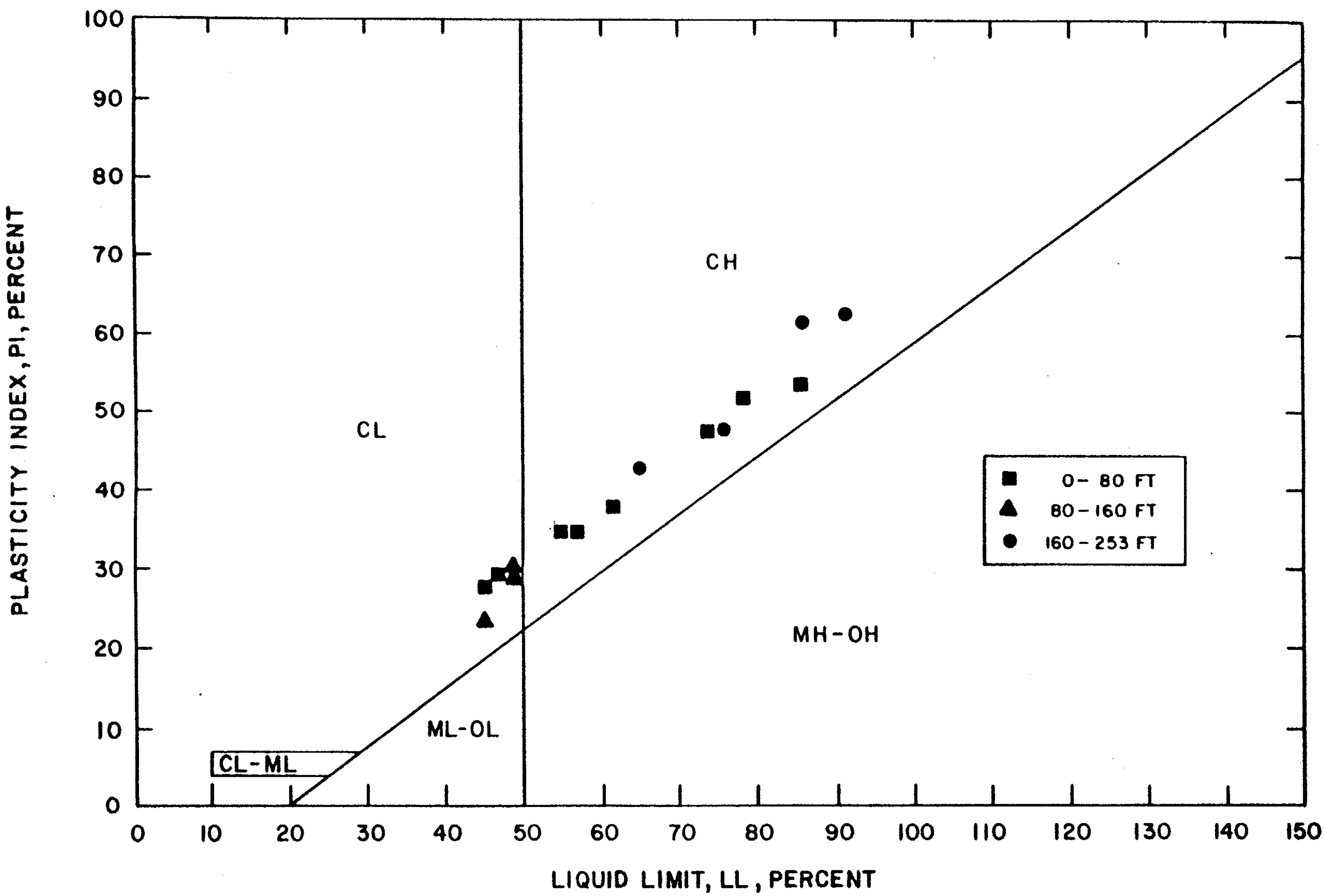


DESIGN SHEAR STRENGTH PROFILE
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO

PLATE 2.4



SUBMERGED UNIT WEIGHT PROFILE
BLOCK 58, WEST DELTA AREA
GULF OF MEXICO

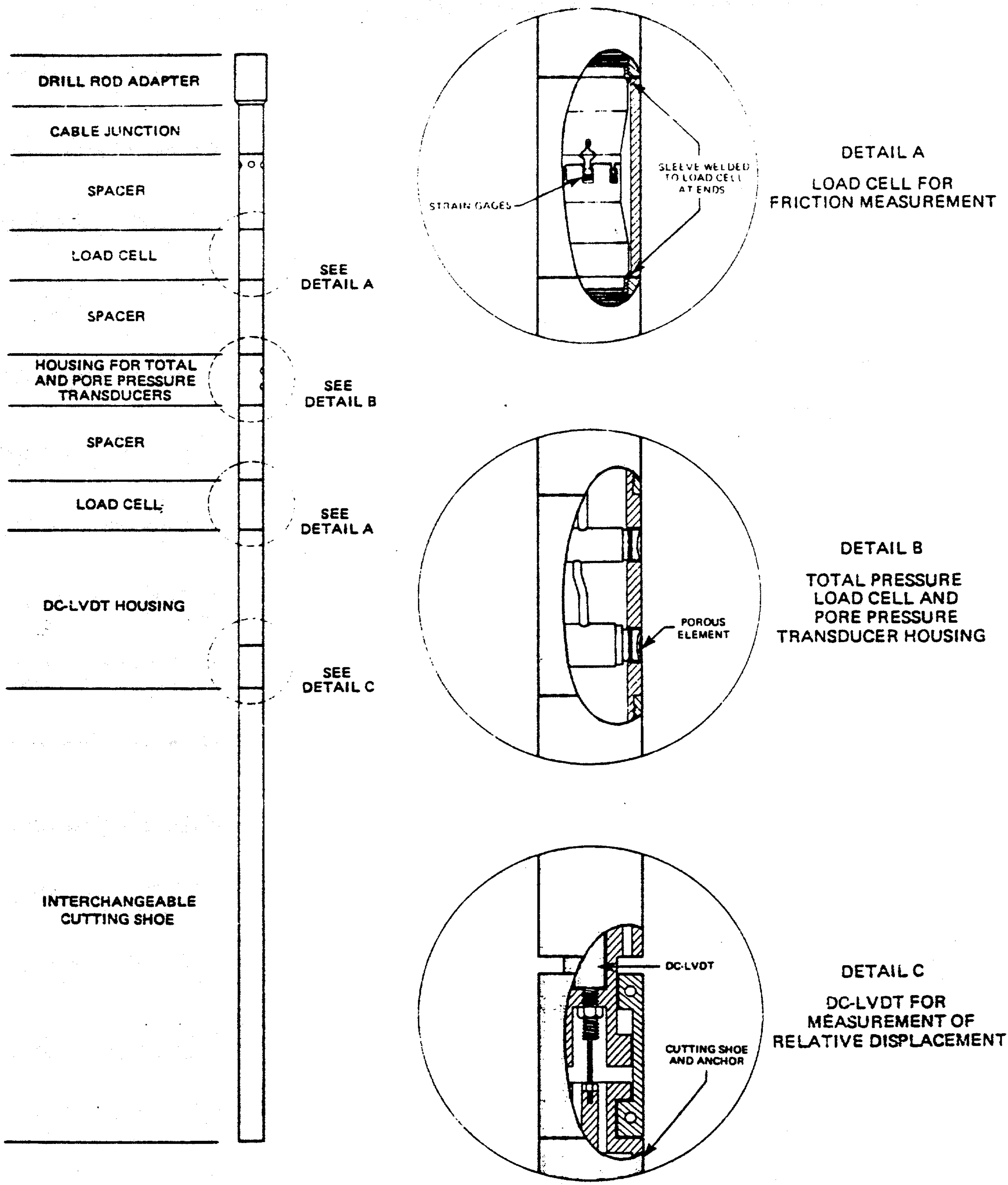


PLASTICITY CHART

BLOCK 58, WEST DELTA AREA

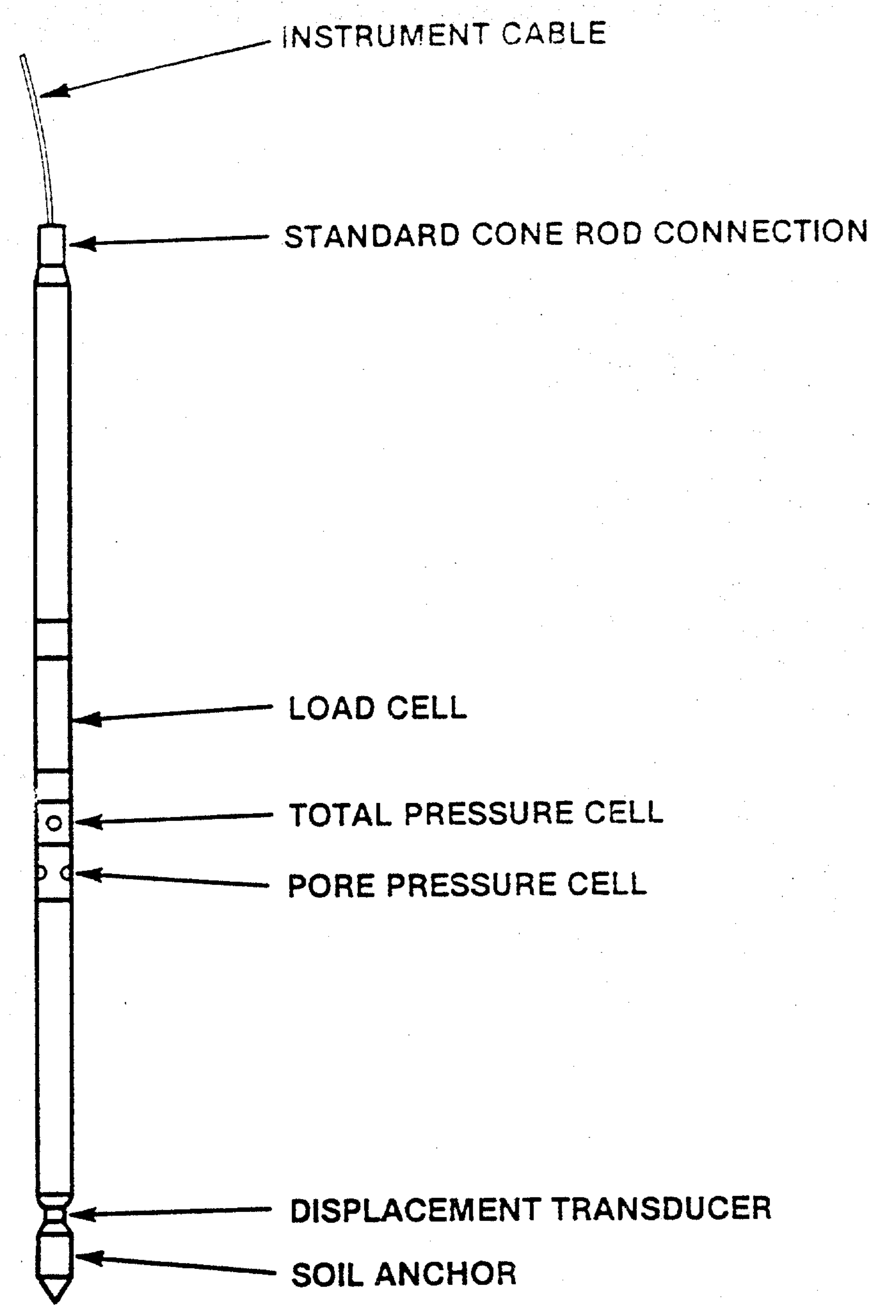
APPENDIX 3

ILLUSTRATIONS FOR CHAPTER 3



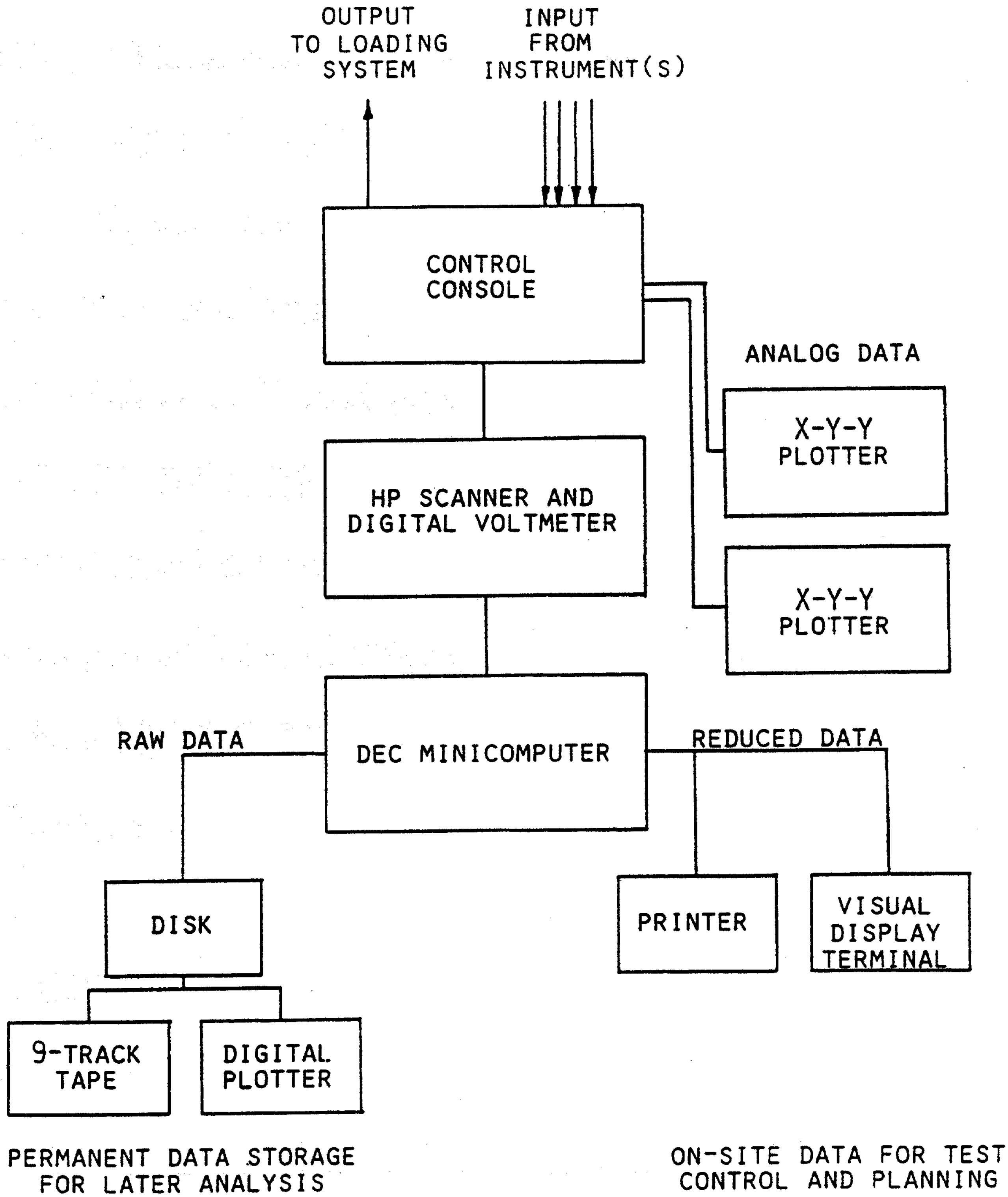
THE 3-INCH DIAMETER PILE SEGMENT MODEL

PLATE 3.1



THE 1.72-INCH DIAMETER PILE SEGMENT MODEL

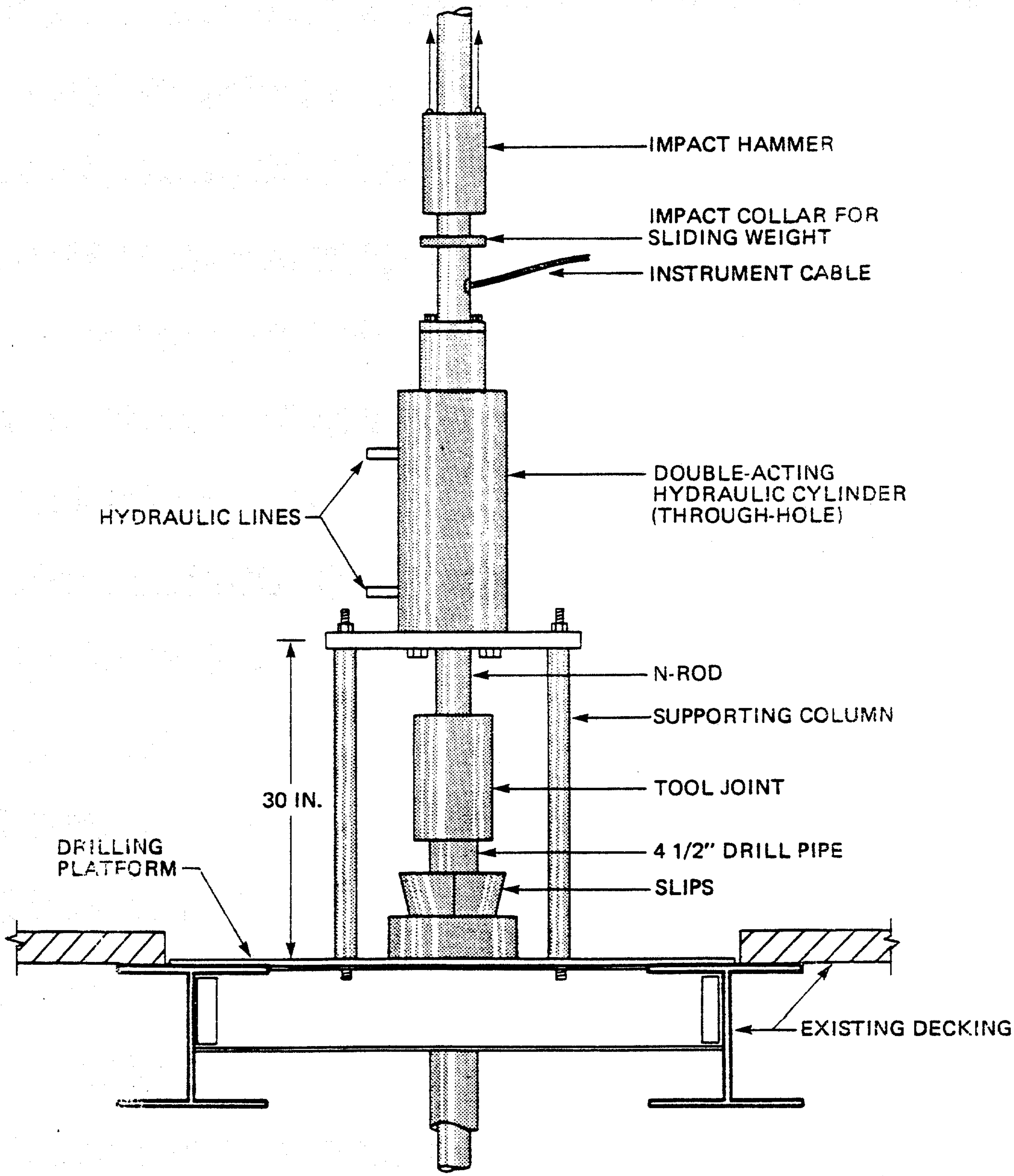
PLATE 3.2

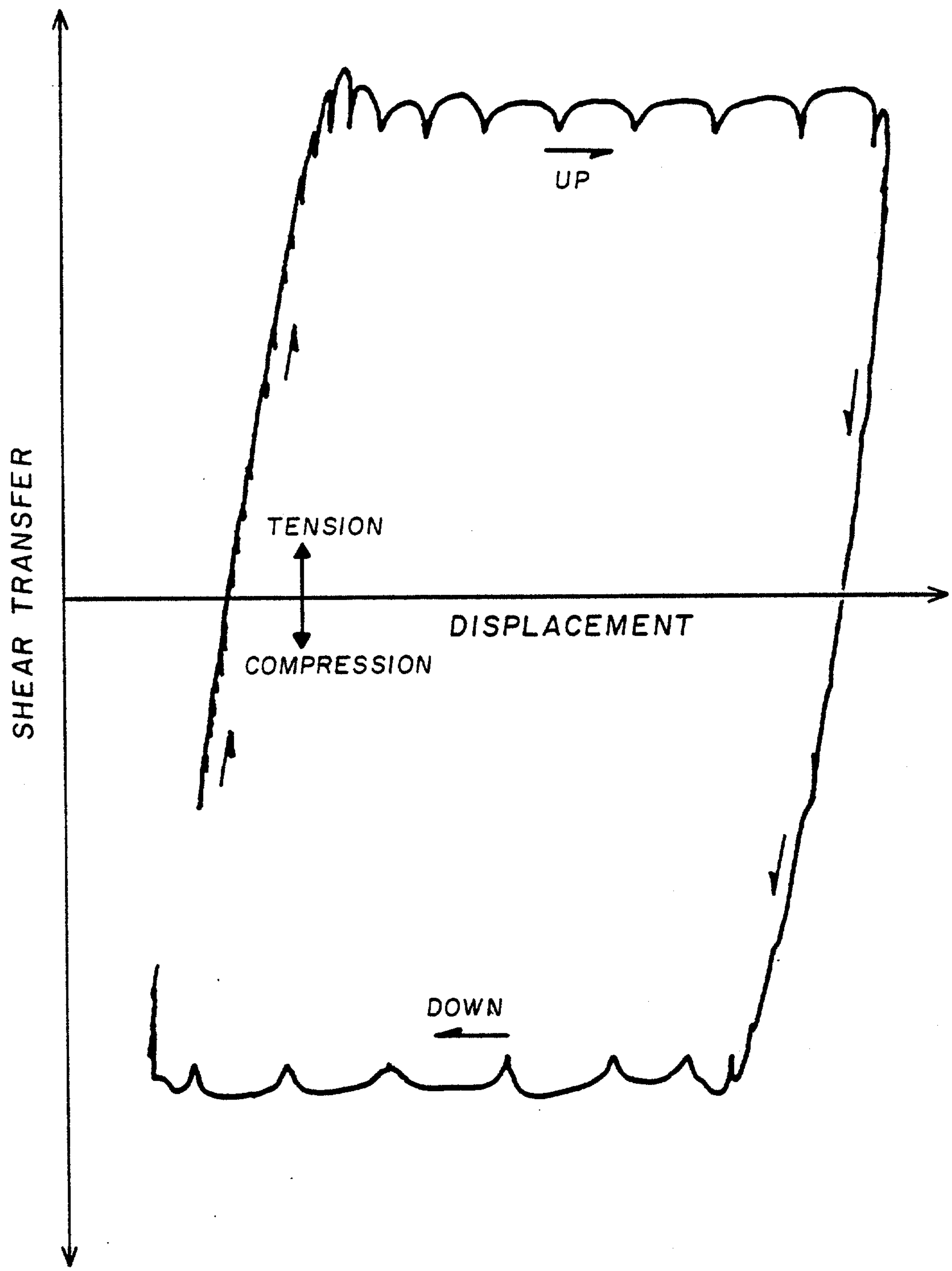


DATA ACQUISITION SYSTEM

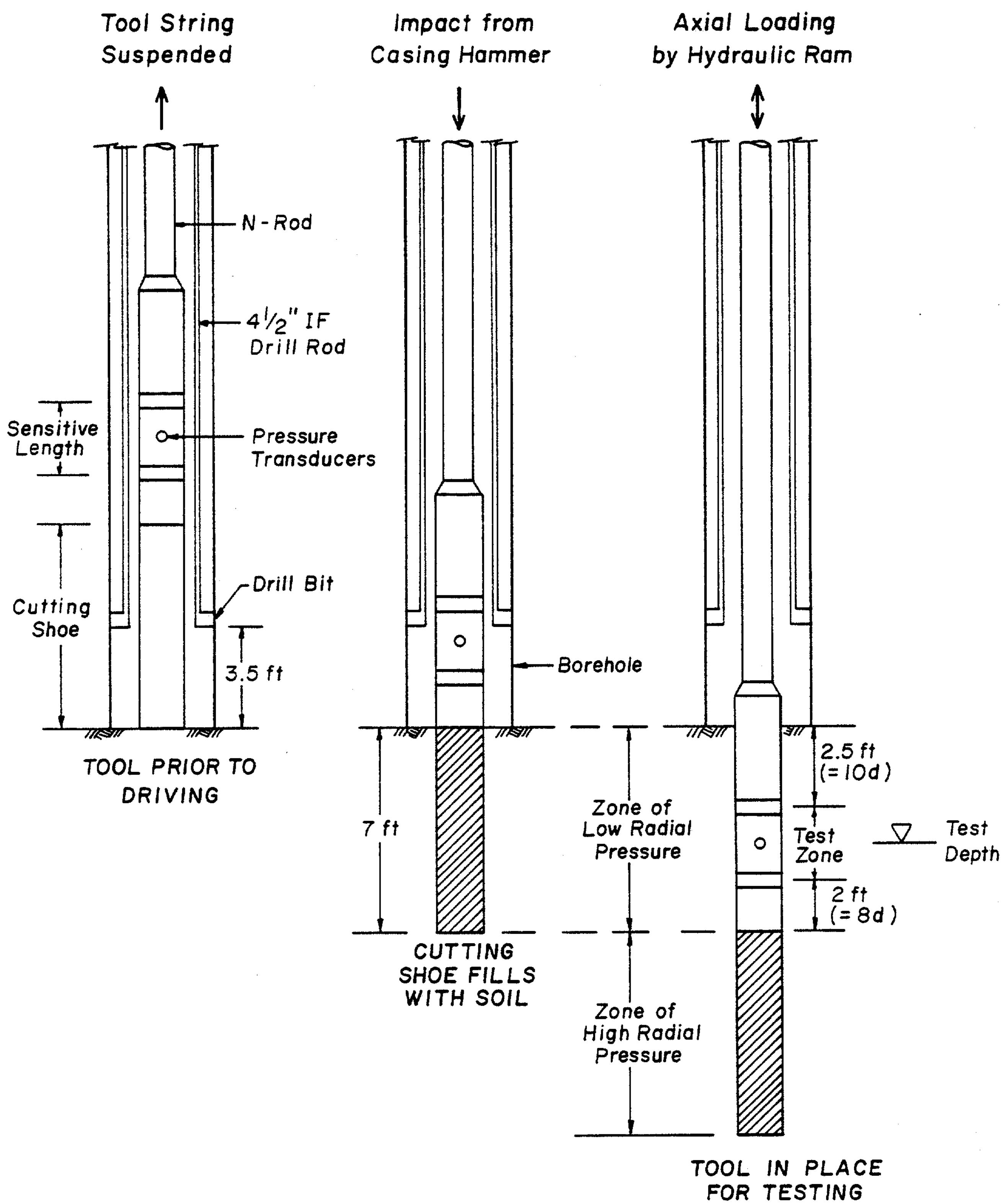
PLATE 3.3

SCHEMATIC DIAGRAM OF TESTING ARRANGEMENT FOR 3-INCH DIAMETER PILE SEGMENT TESTS





EFFECTS OF PLATFORM MOTION ON CYCLIC LOAD TESTS



PHASES OF INSTALLATION OF THE 3-INCH DIAMETER PILE SEGMENT MODEL

APPENDIX 4

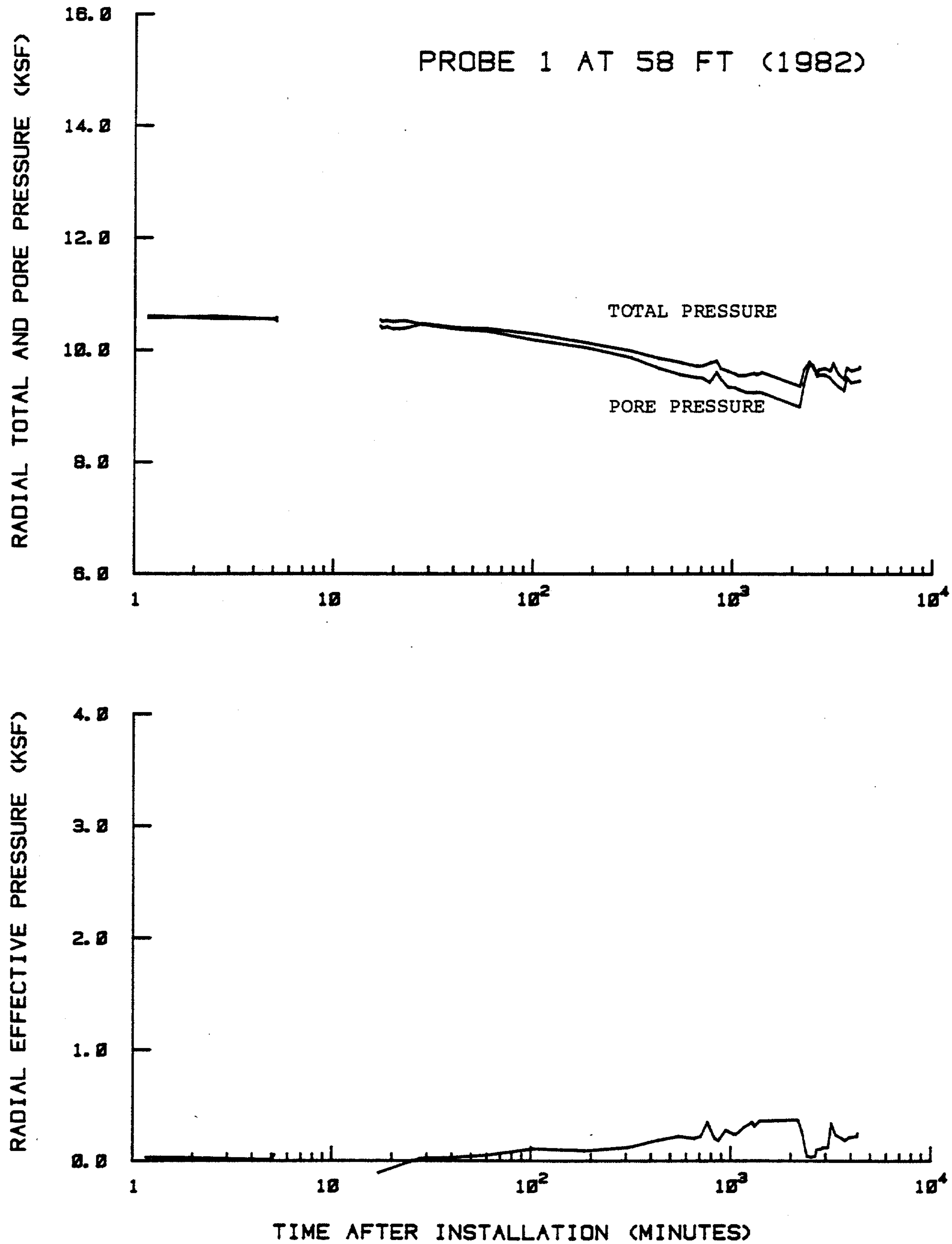
ILLUSTRATIONS FOR CHAPTER 4

NO ILLUSTRATIONS FOR CHAPTER 4

APPENDIX 5

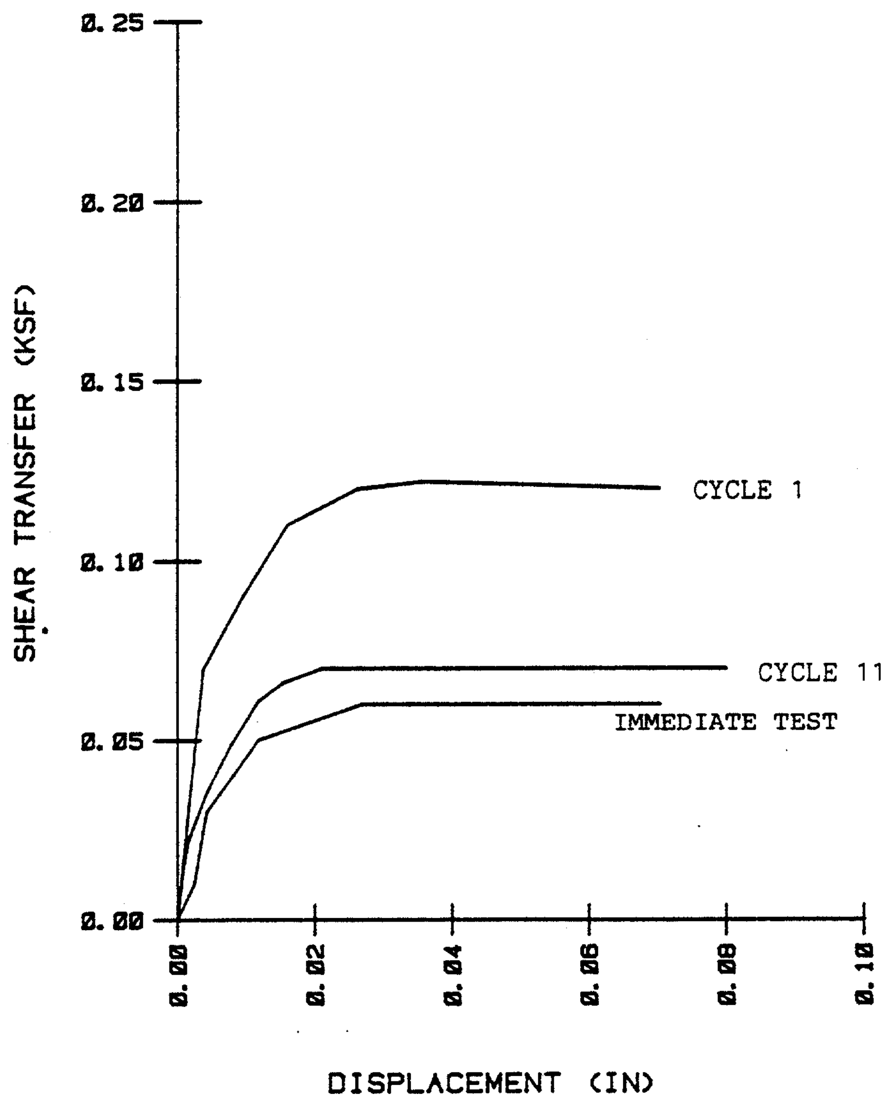
ILLUSTRATIONS FOR CHAPTER 5

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 1 AT 58 FEET

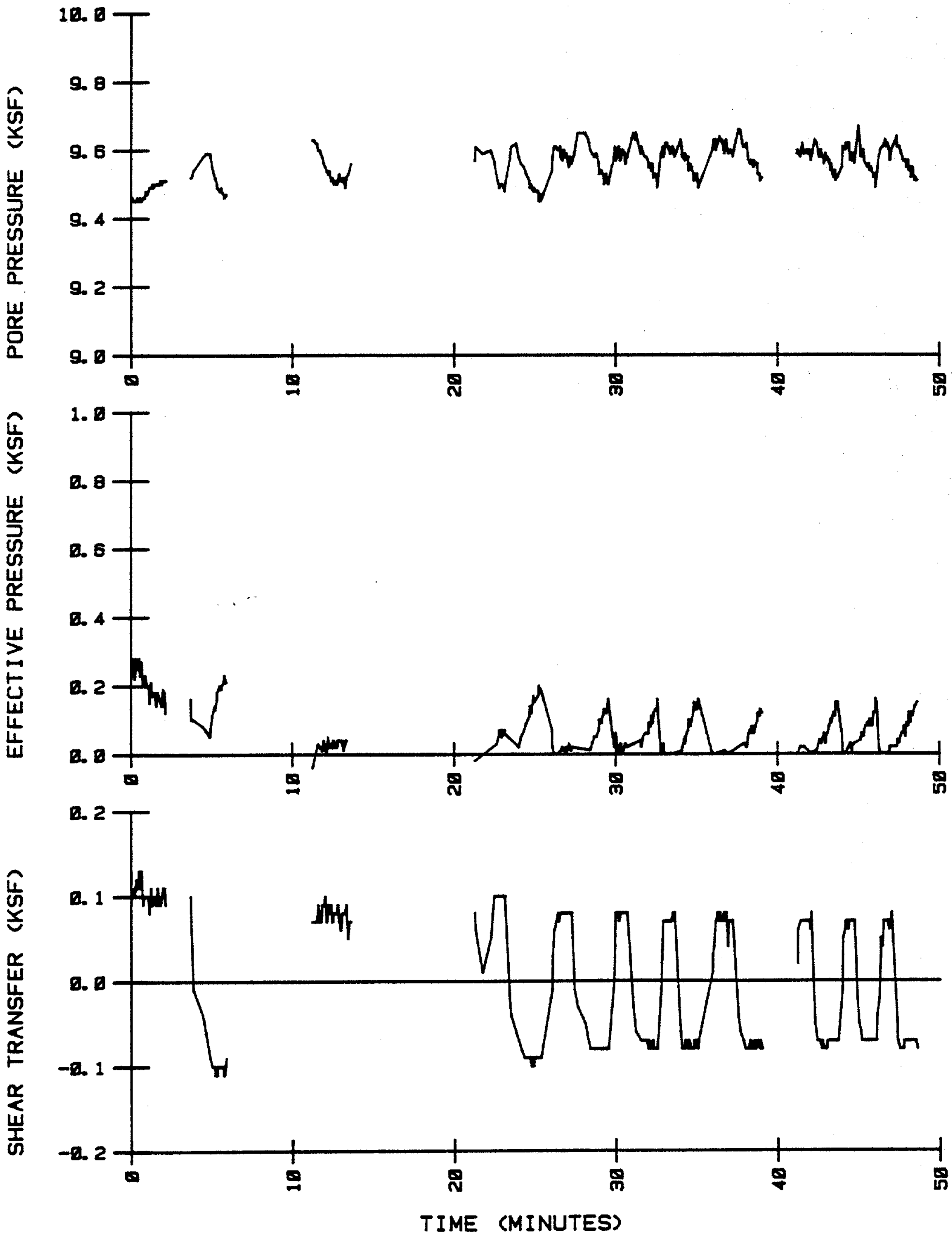
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 58 FT - 72 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 1 AT 58 FEET

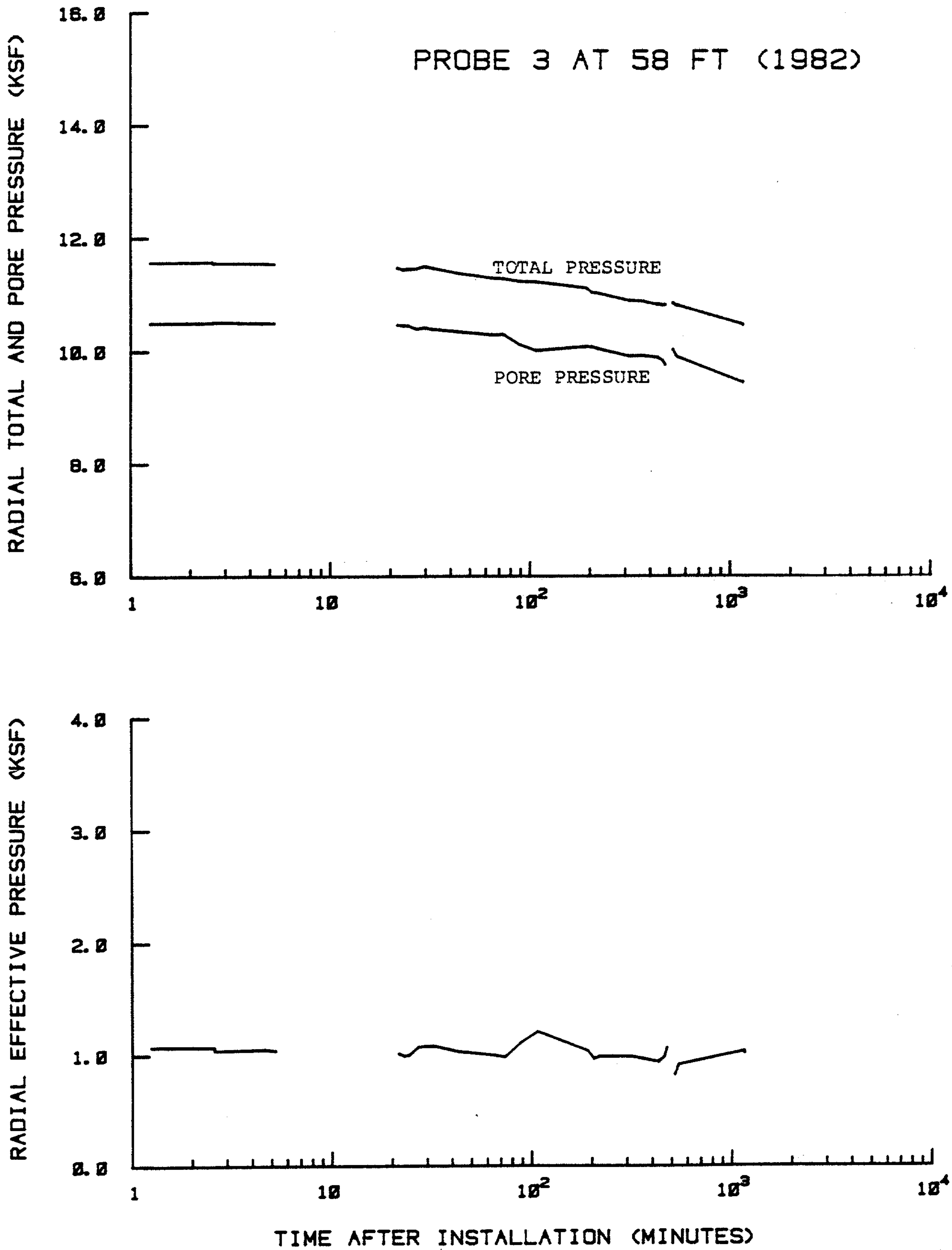
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 58 FT - 72 HR TEST

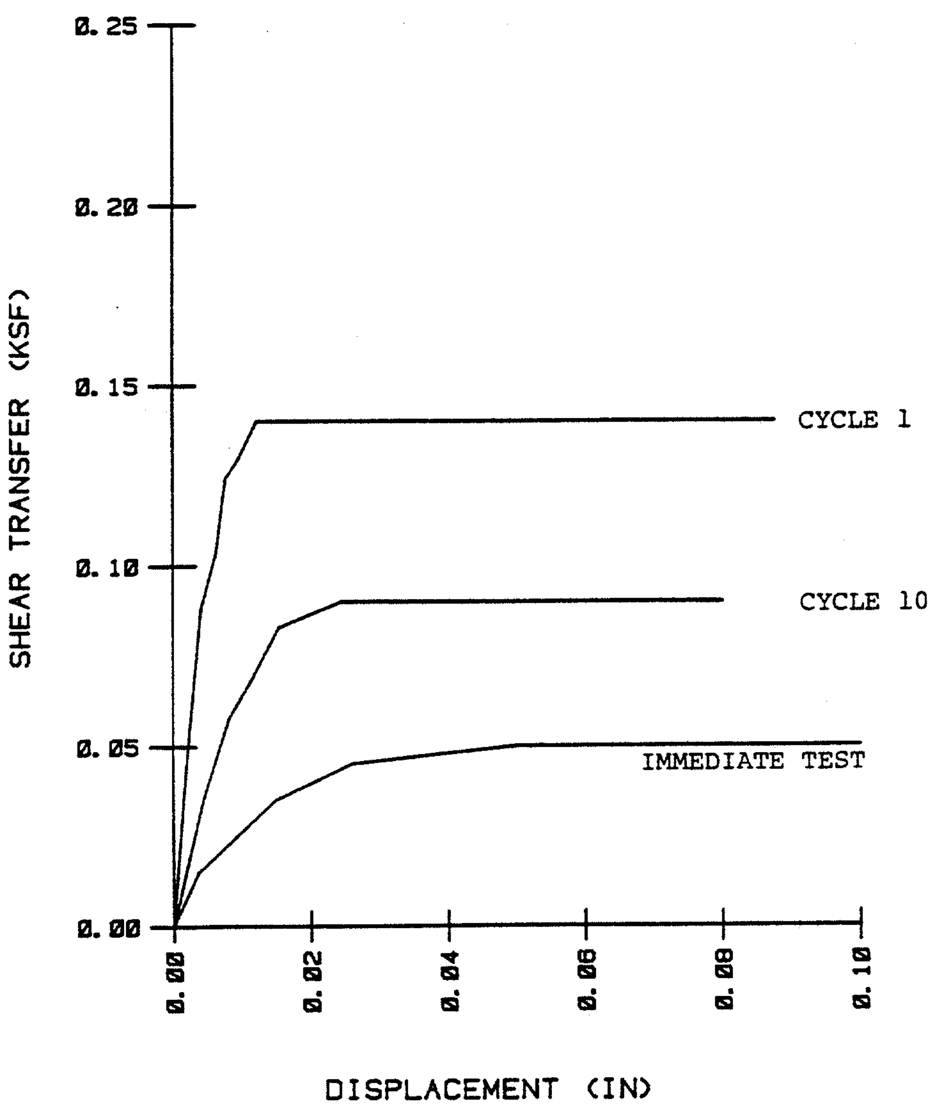
SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 1 AT 58 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 2 AT 58 FEET

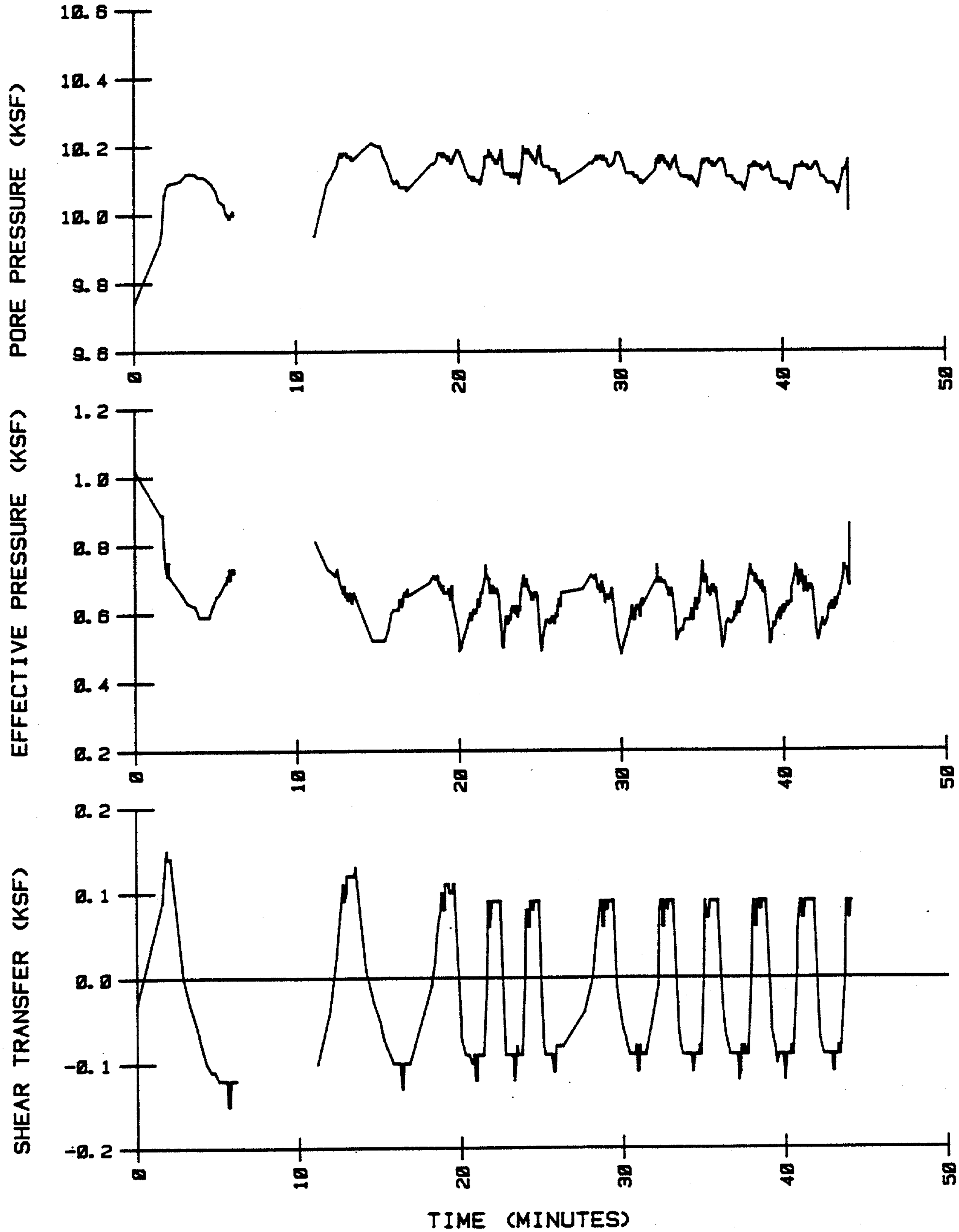
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 58 FT - 8 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 2 AT 58 FEET

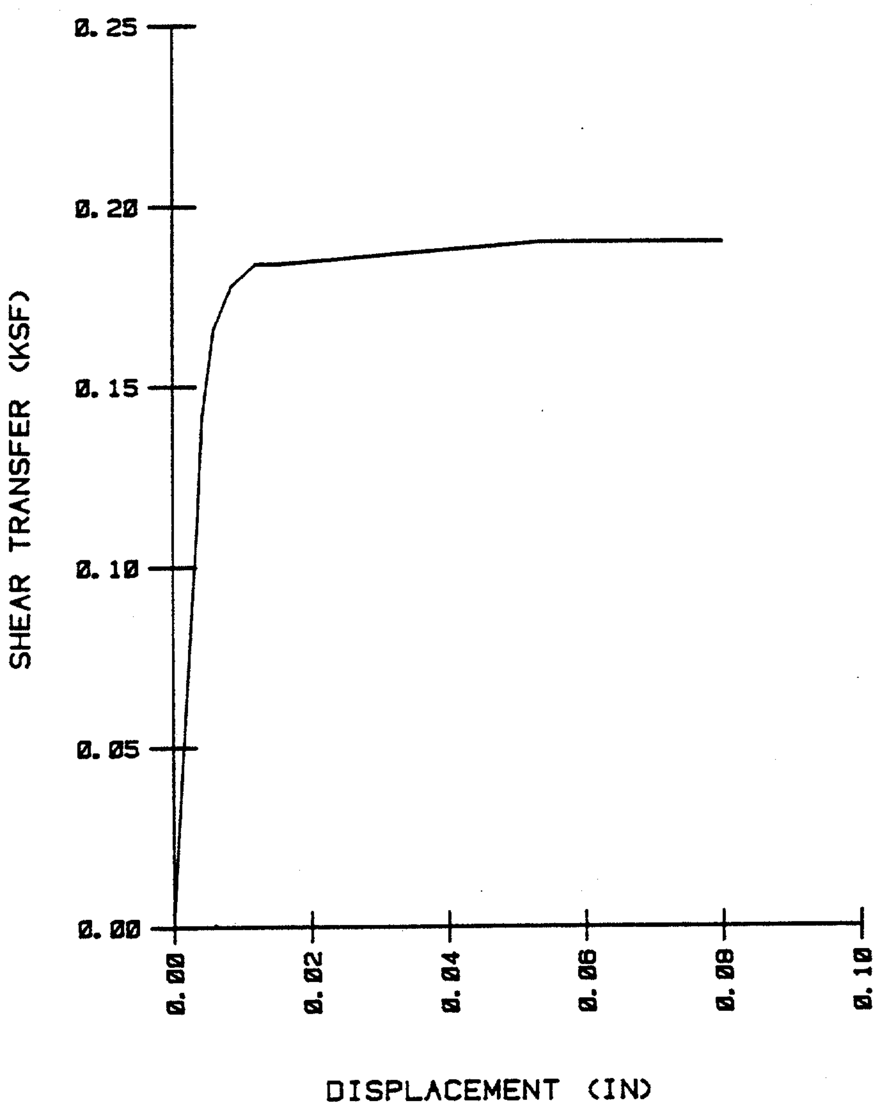
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 58 FT - 8 HR TEST

SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 2 AT 58 FEET

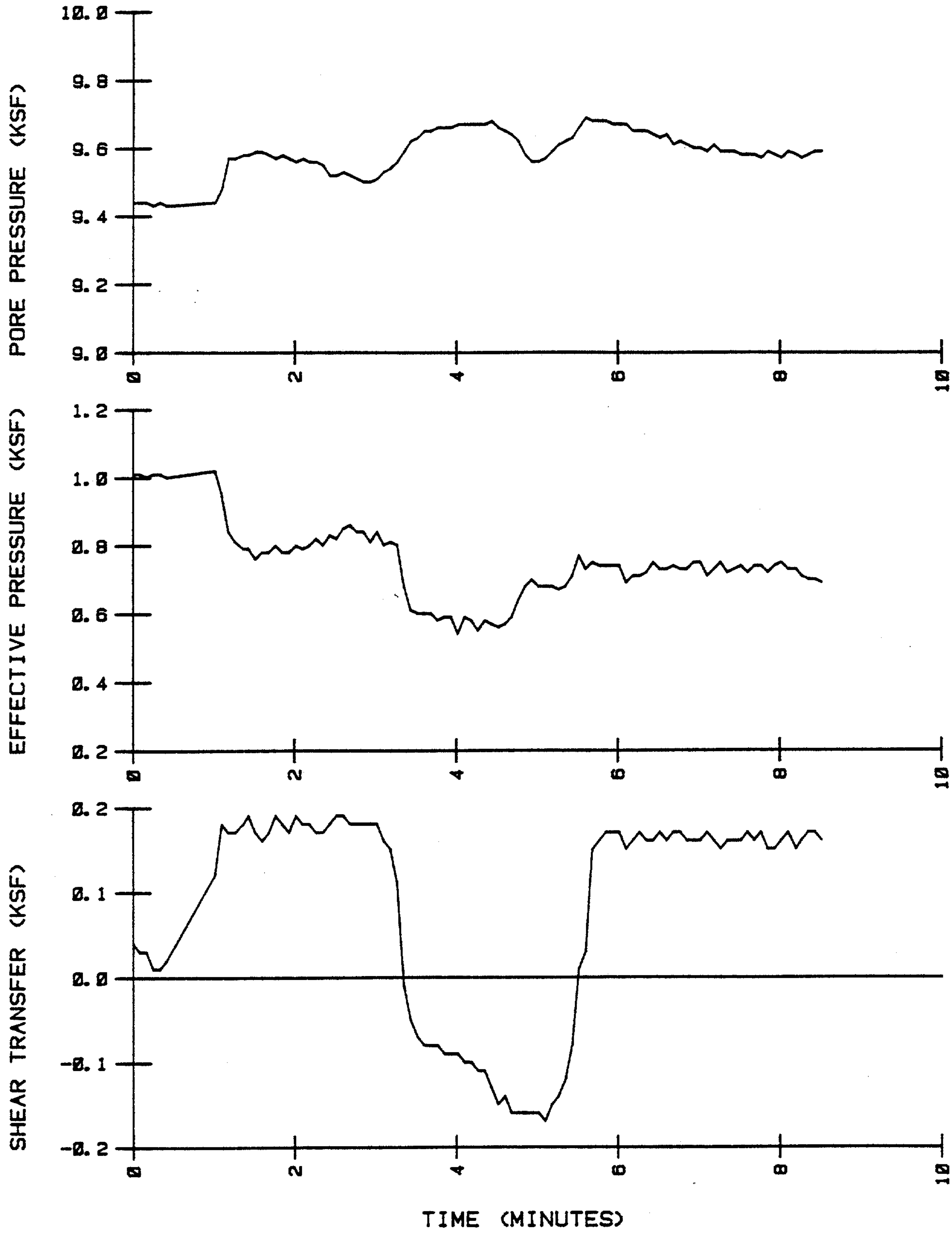
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 58 FT - PULL OUT TEST

SHEAR TRANSFER CURVE RECORDED AFTER ADDITIONAL CONSOLIDATION IN EXPERIMENT 2 AT 58 FEET

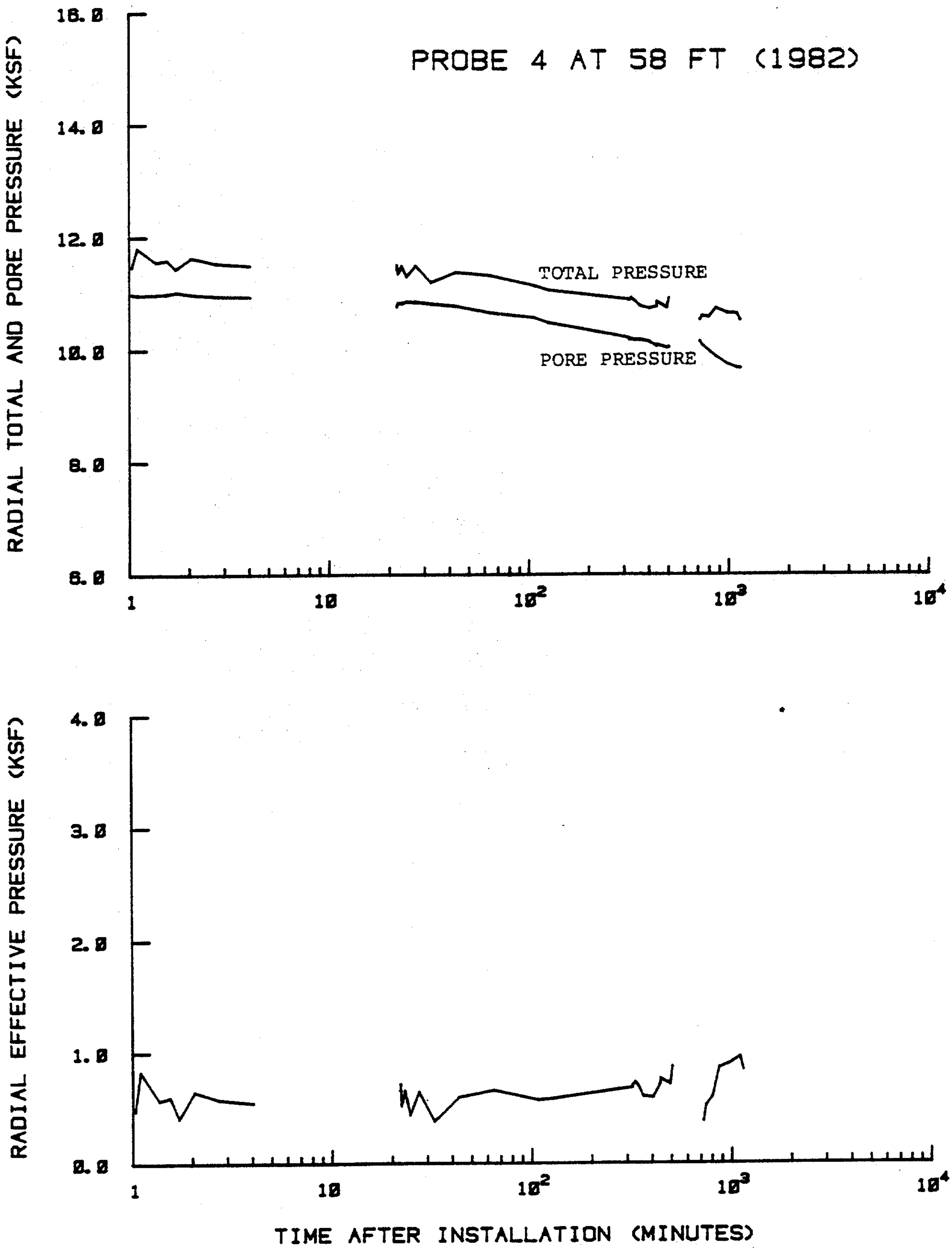
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 58 FT - PULL OUT TEST

SOIL BEHAVIOR RECORDED DURING THE RETEST AFTER ADDITIONAL CONSOLIDATION IN EXPERIMENT 2 AT 58 FEET

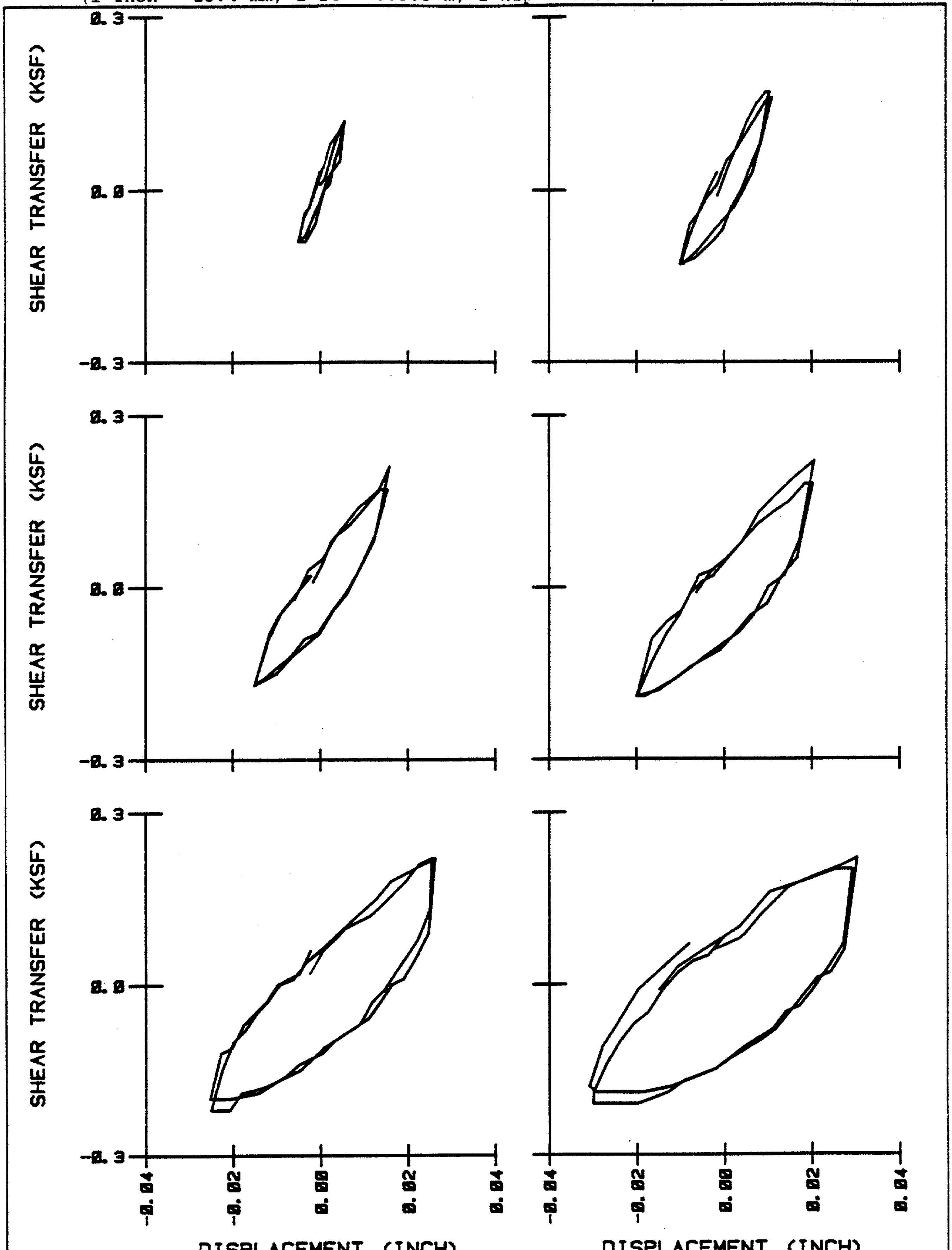
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 3 AT 58 FEET

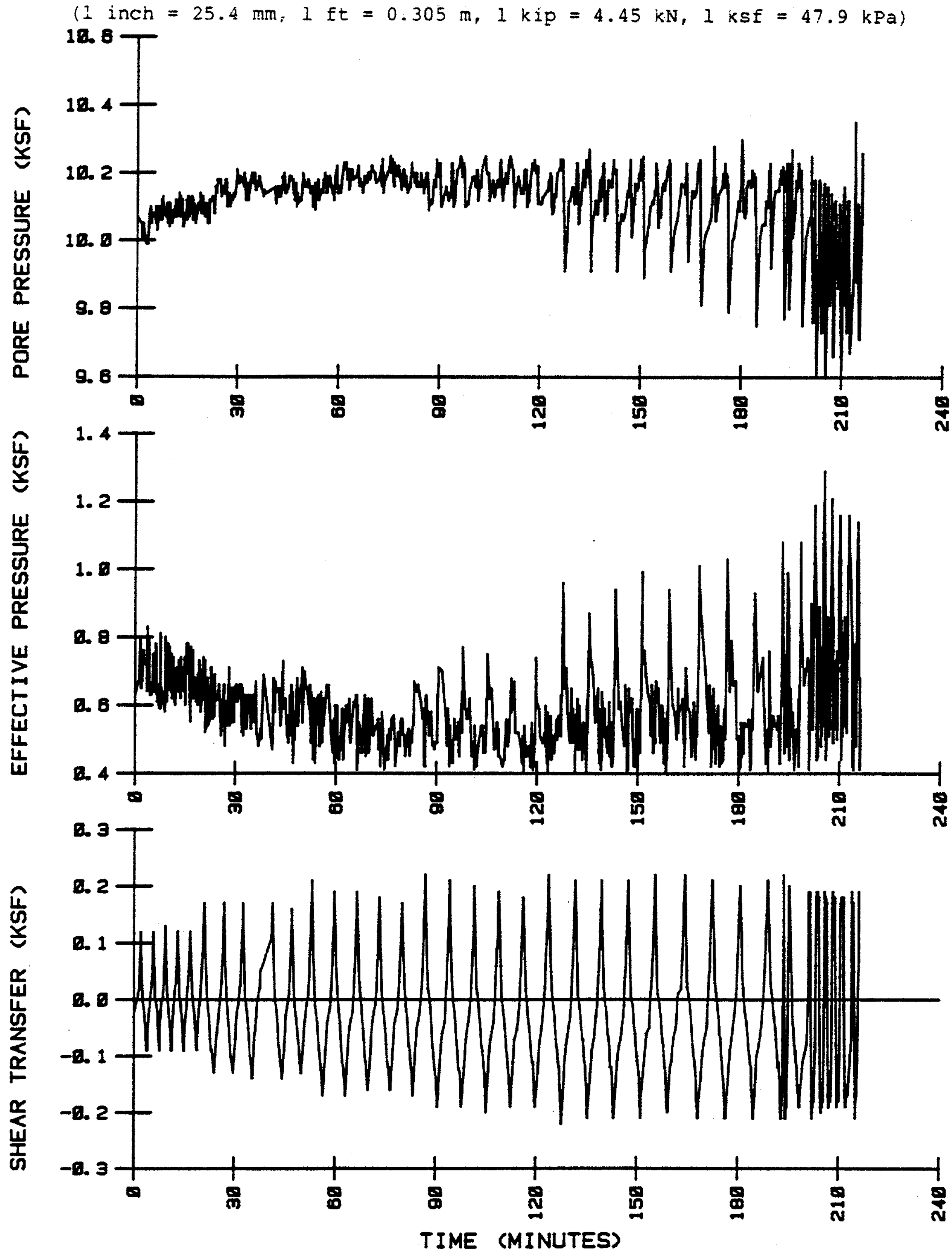
PLATE 5.9

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



EVOLUTION OF PLASTIC BEHAVIOR DURING THE SMALL-DISPLACEMENT CYCLIC TESTS IN EXPERIMENT 3 AT 58 FEET

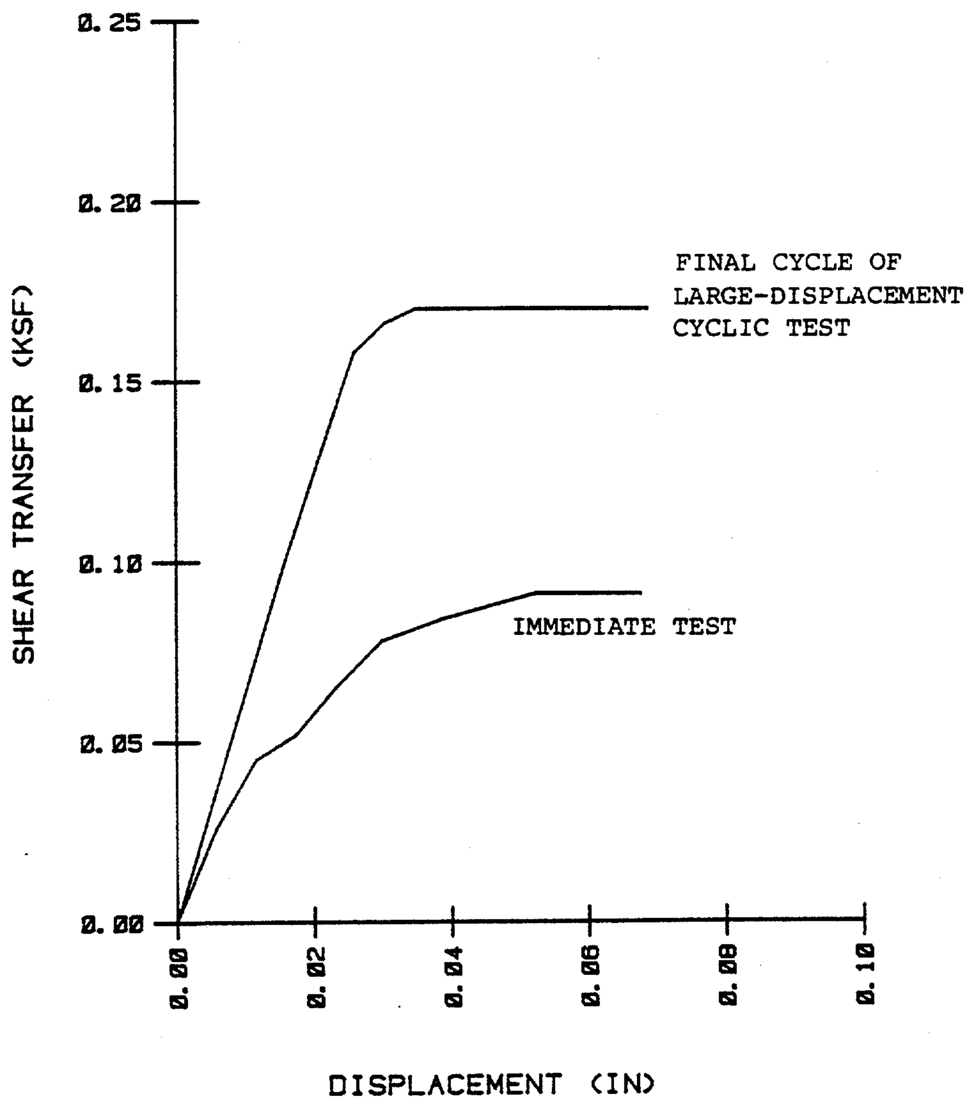
PLATE 5.10



PROBE 4 AT 58 FT - 8 HR TEST

SOIL BEHAVIOR RECORDED DURING THE SMALL-DISPLACEMENT CYCLIC TESTS IN EXPERIMENT 3 AT 58 FEET

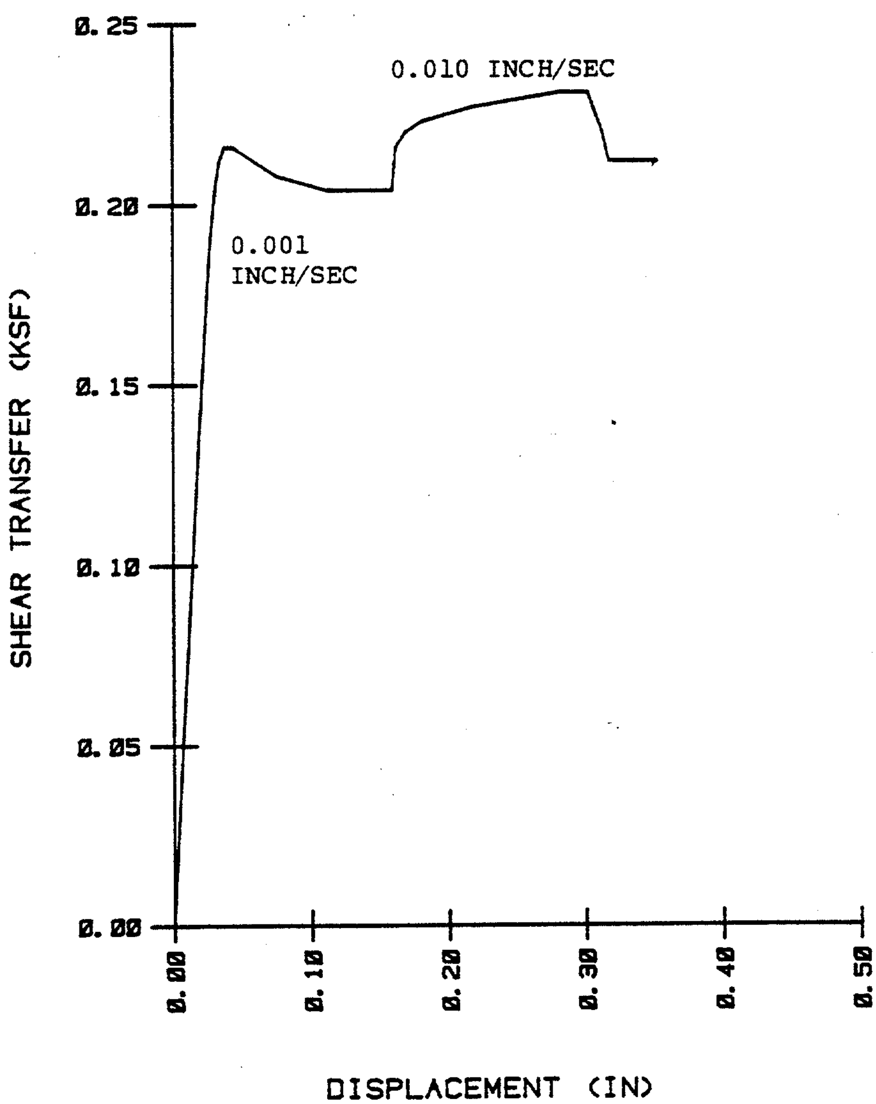
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 58 FT - 8 HR TEST

CYCLIC MINIMUM SHEAR TRANSFER CURVE RECORDED IN EXPERIMENT 3 AT 58 FEET

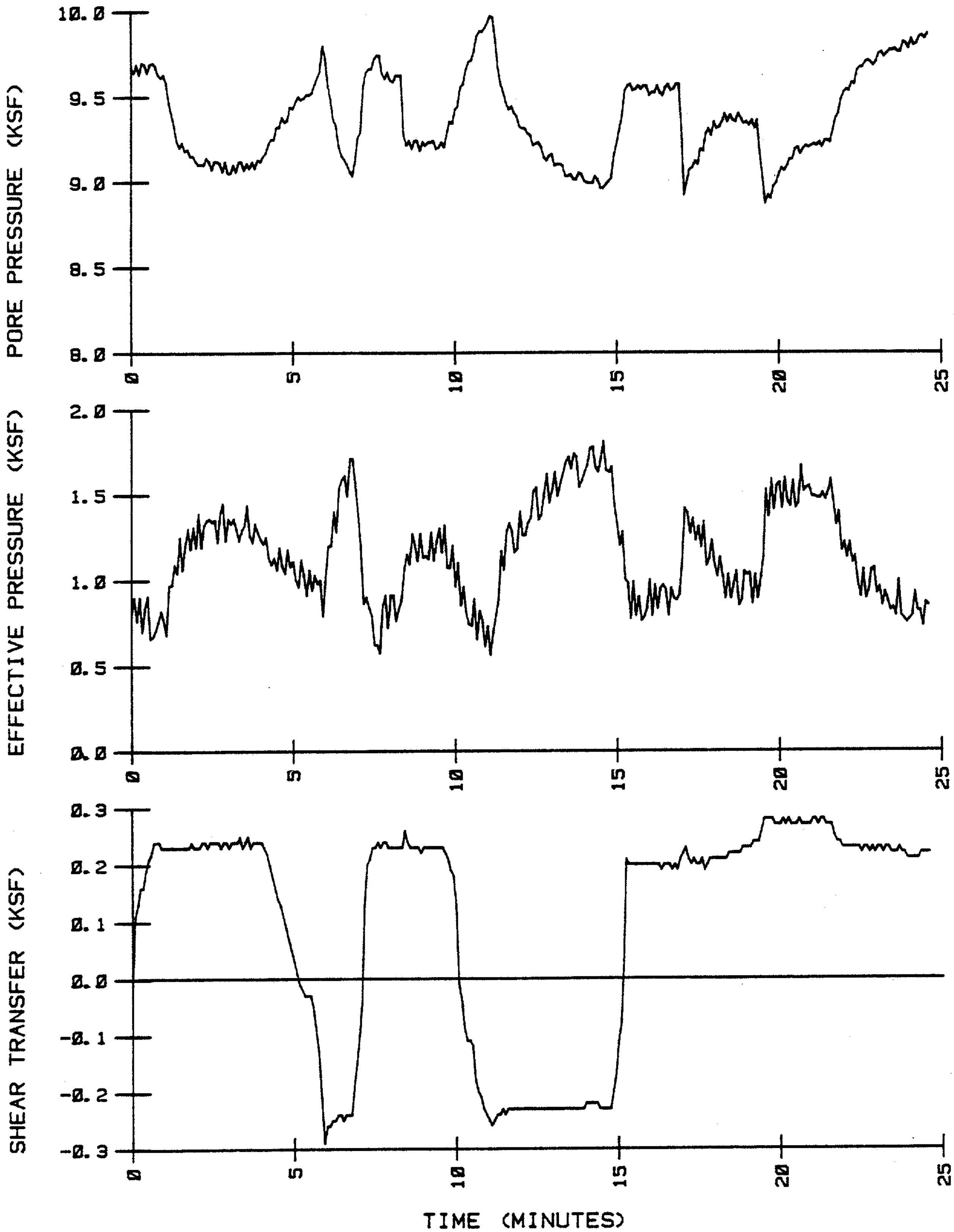
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 58 FT - PULL OUT TEST

SHEAR TRANSFER CURVE RECORDED AFTER ADDITIONAL CONSOLIDATION IN EXPERIMENT 3 AT 58 FEET

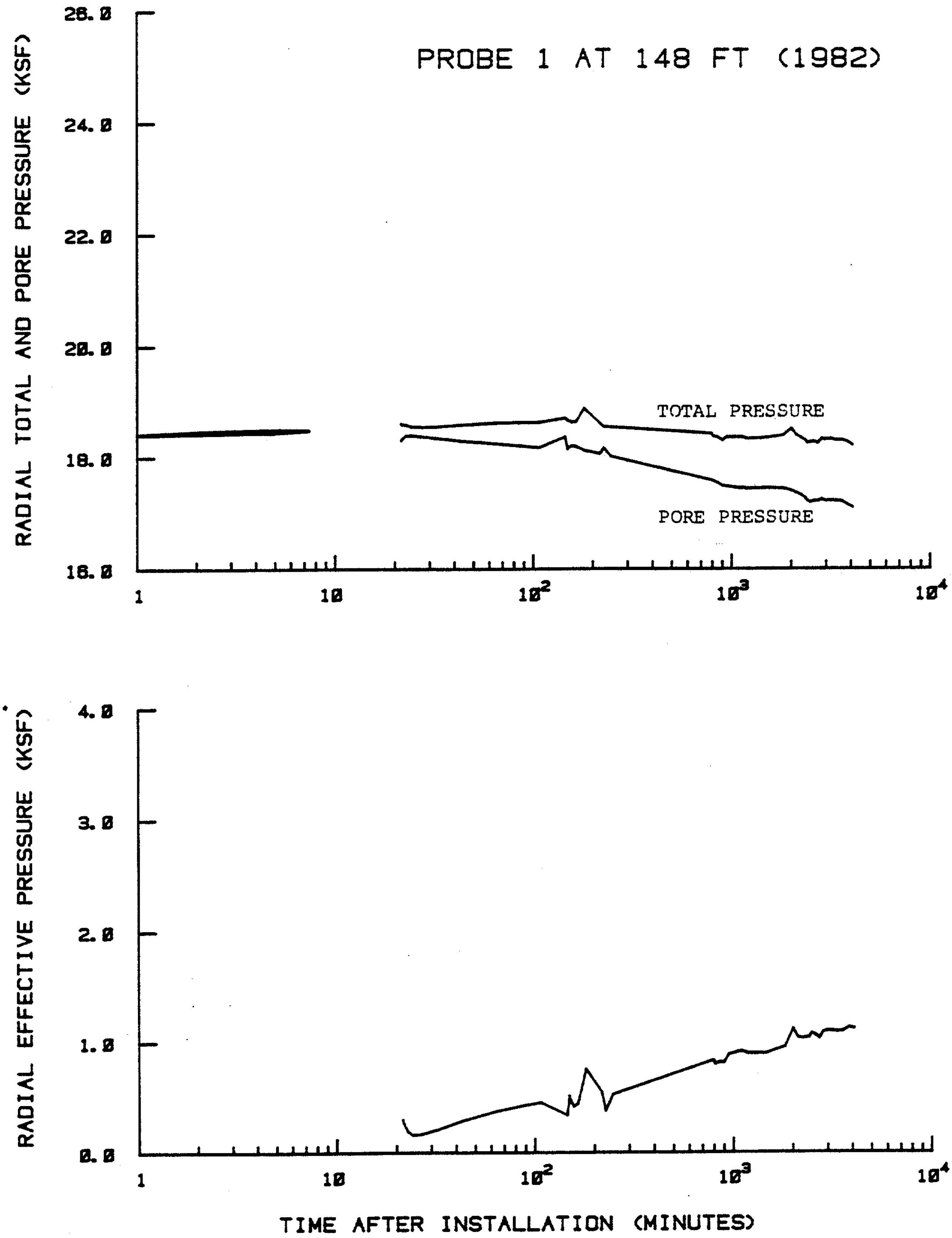
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 58 FT - PULL OUT TEST

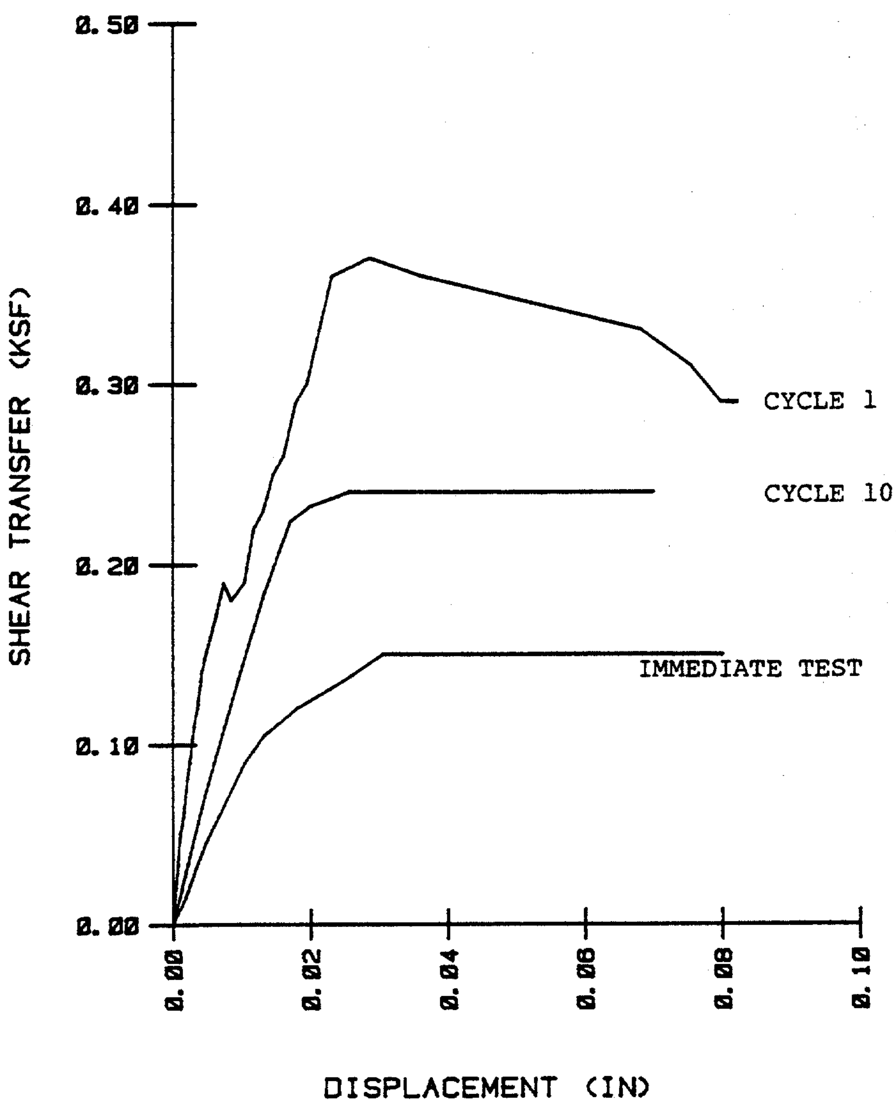
SOIL BEHAVIOR RECORDED DURING THE RETEST AFTER ADDITIONAL CONSOLIDATION IN EXPERIMENT 3 AT 58 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



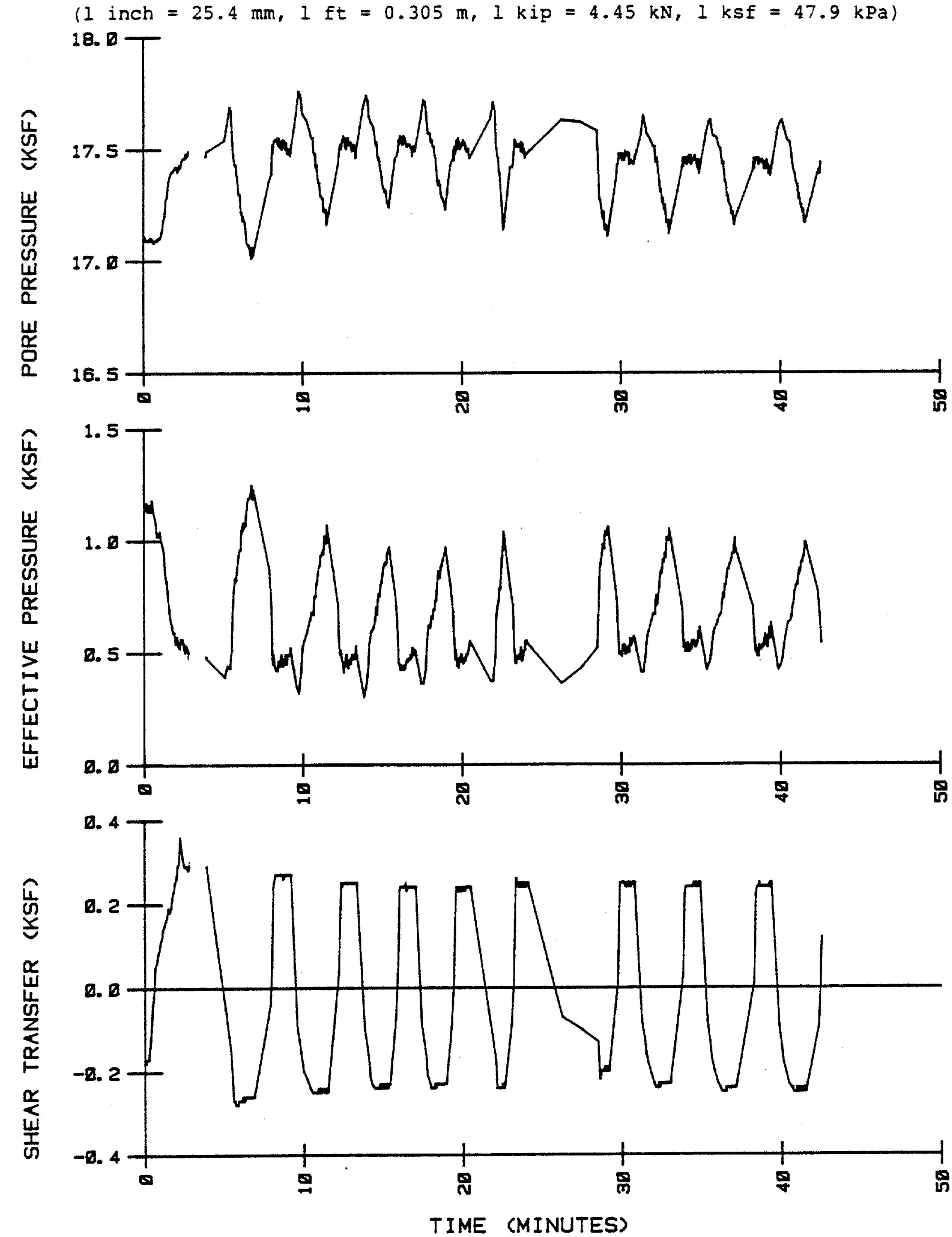
SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 1 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 148 FT - 68 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 3 AT 148 FEET

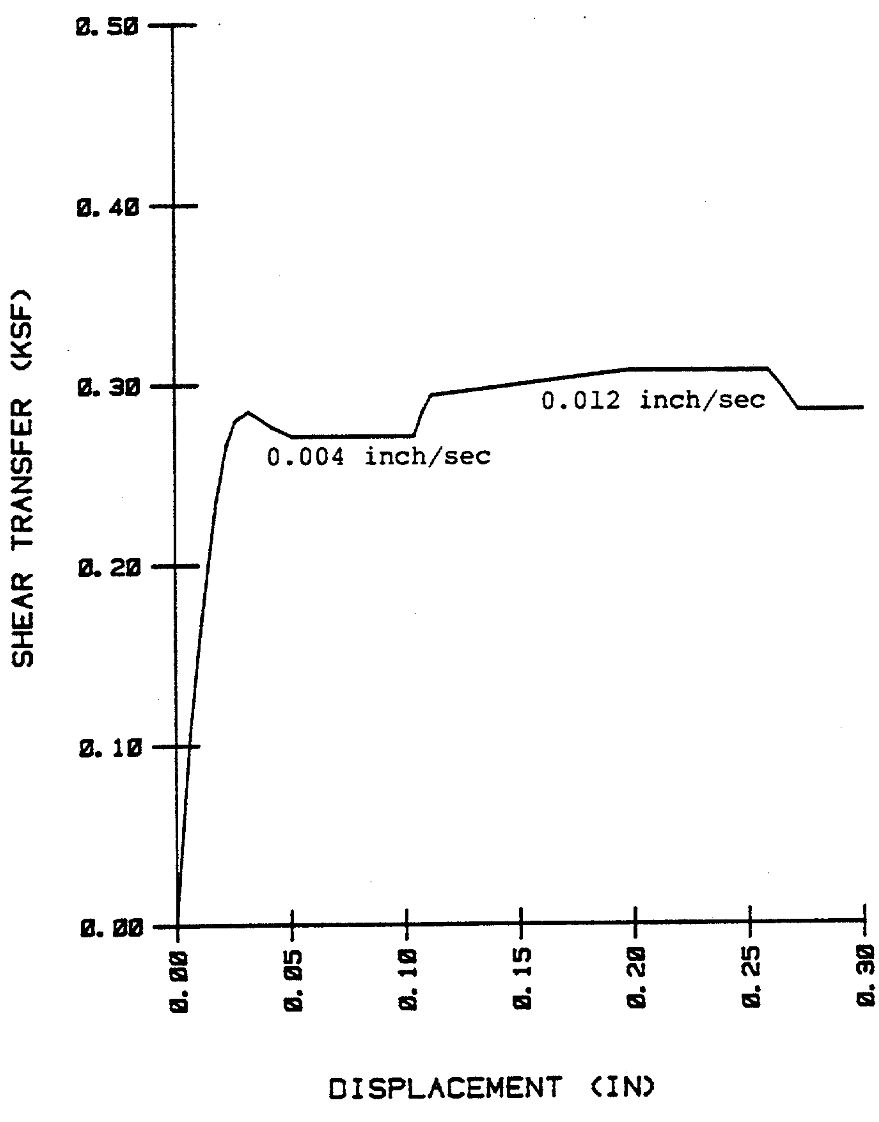


PROBE 1 AT 148 FT - 68 HR TEST

SOIL BEHAVIOR RECORDED DURING THE INITIAL
CYCLIC TEST IN EXPERIMENT 3 AT 148 FEET

PLATE 5.17

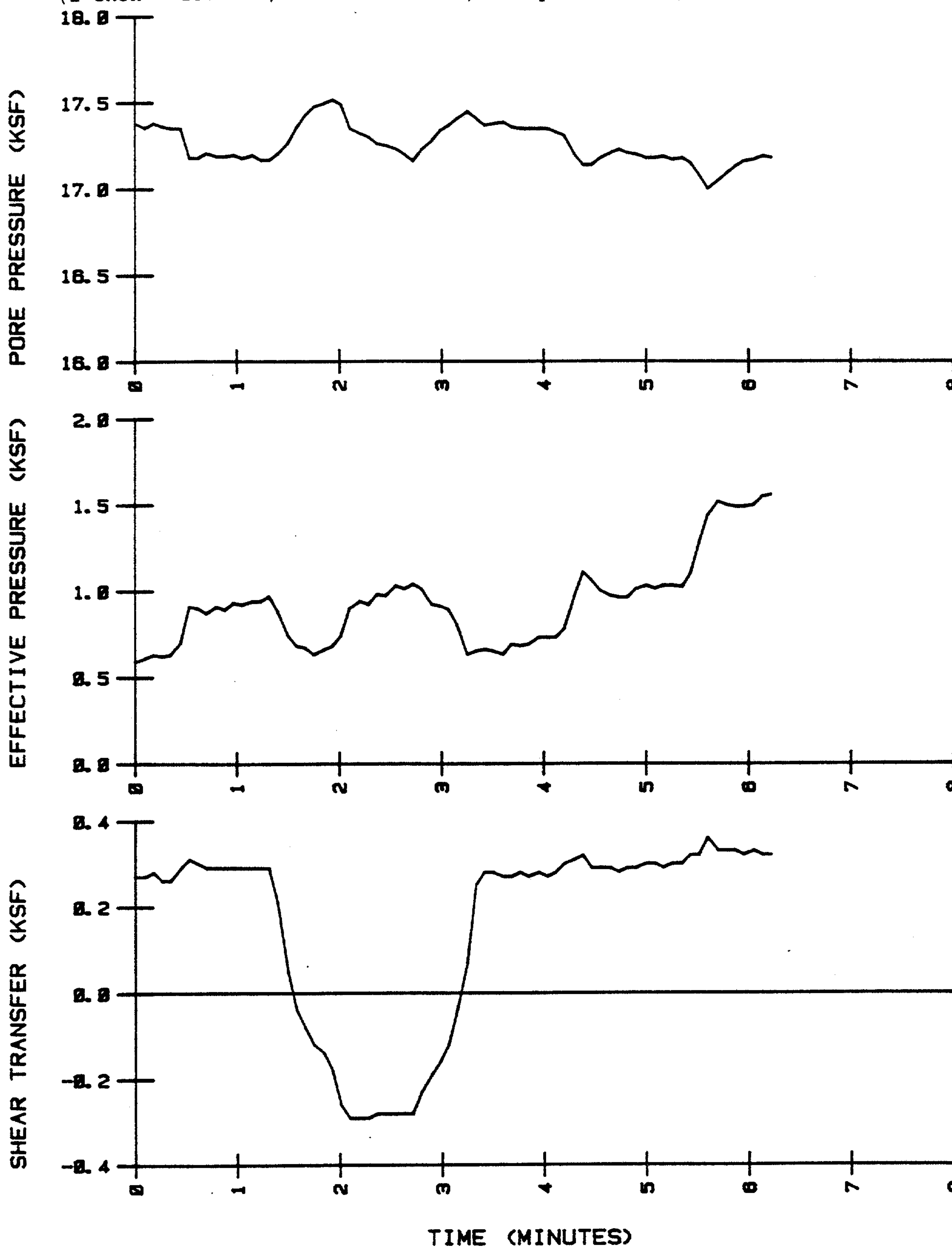
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 148 FT - PULL OUT TEST

EFFECTS OF LOAD RATE ON THE SHEAR TRANSFER RECORDED DURING EXPERIMENT 3 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)

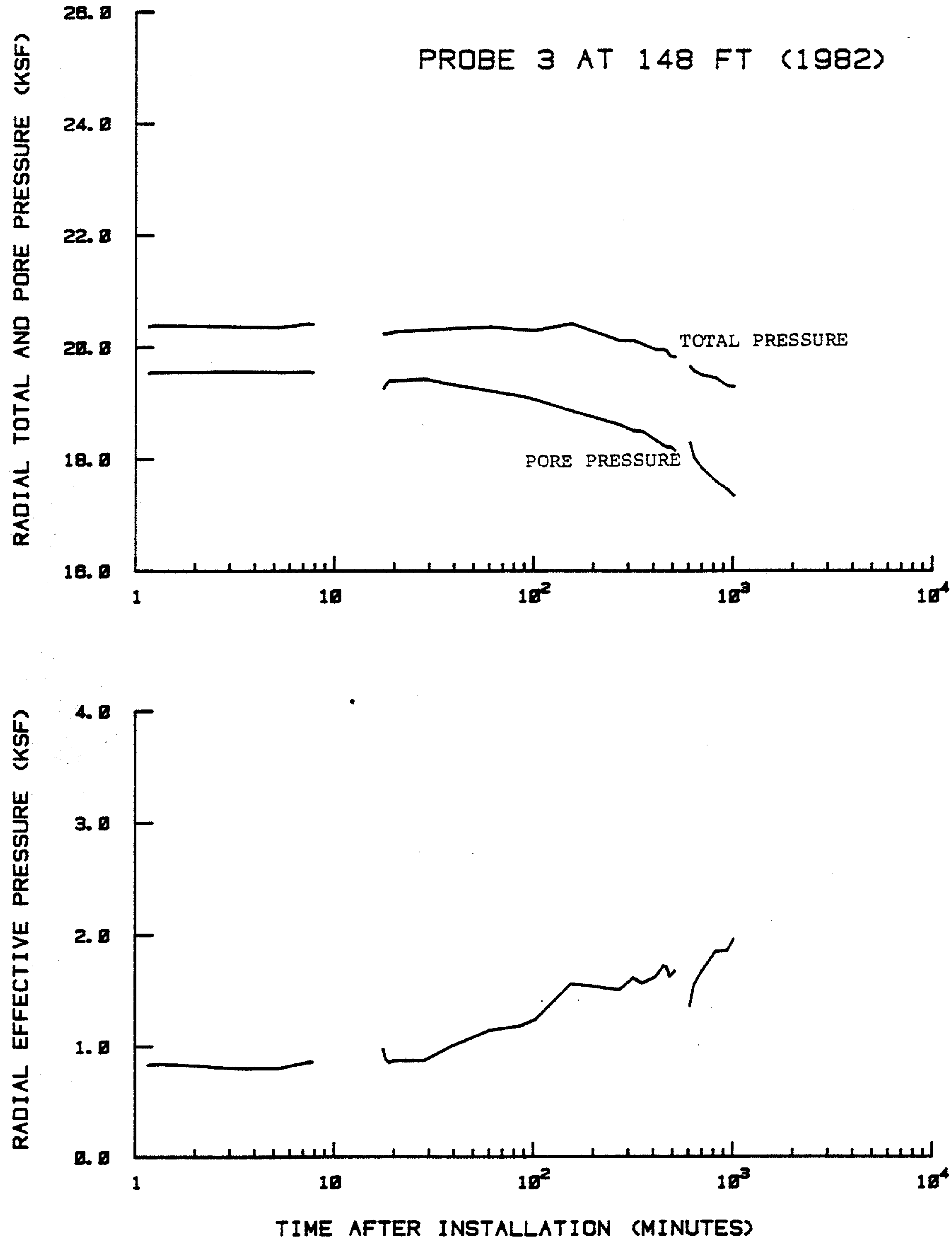


PROBE 1 AT 148 FT - PULL OUT TEST

EFFECTS OF LOAD RATE ON THE SOIL PRESSURES RECORDED
DURING EXPERIMENT 3 AT 148 FEET

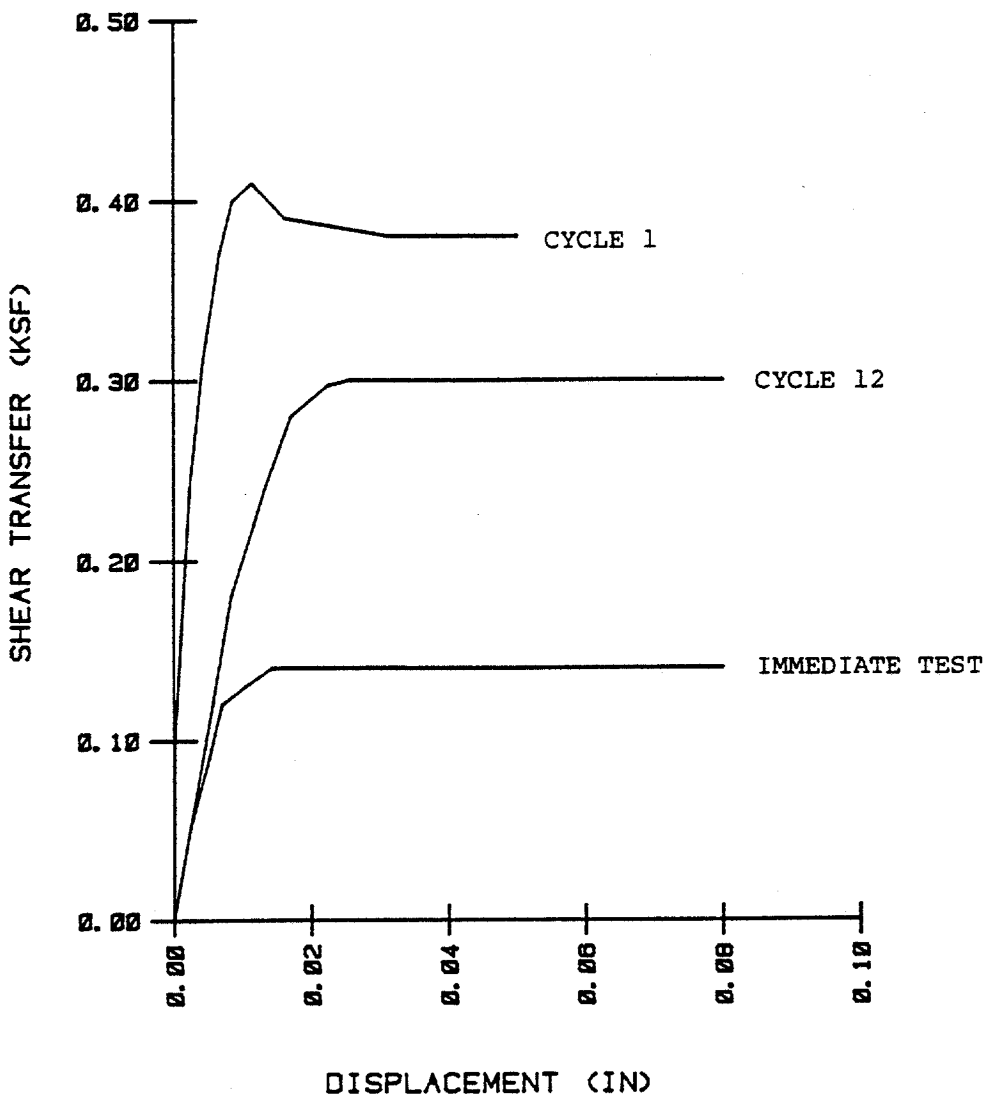
PLATE 5.19

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 2 AT 148 FEET

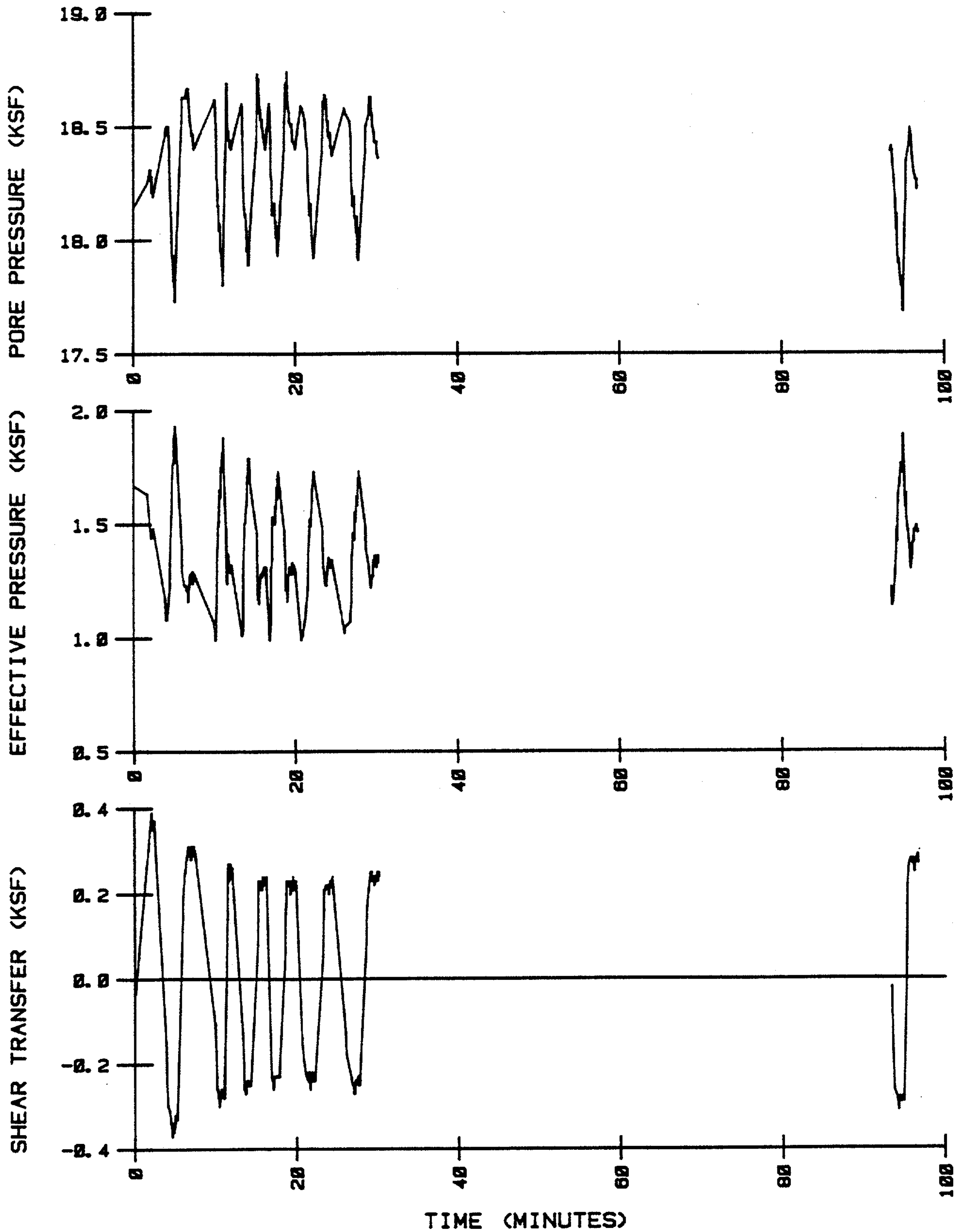
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 148 FT - 8 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 2 AT 148 FEET

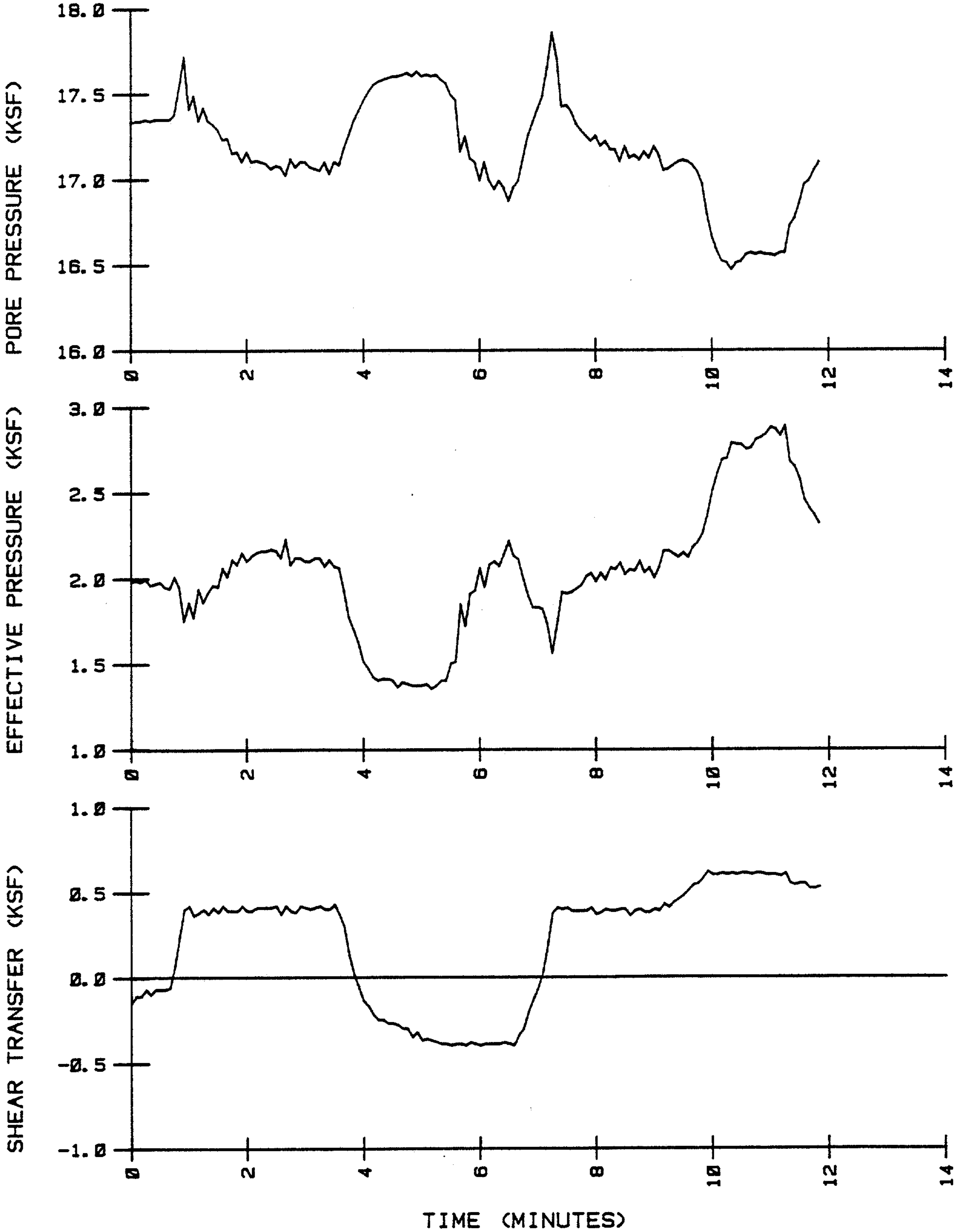
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 148 FT - 8 HR TEST

SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 2 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)

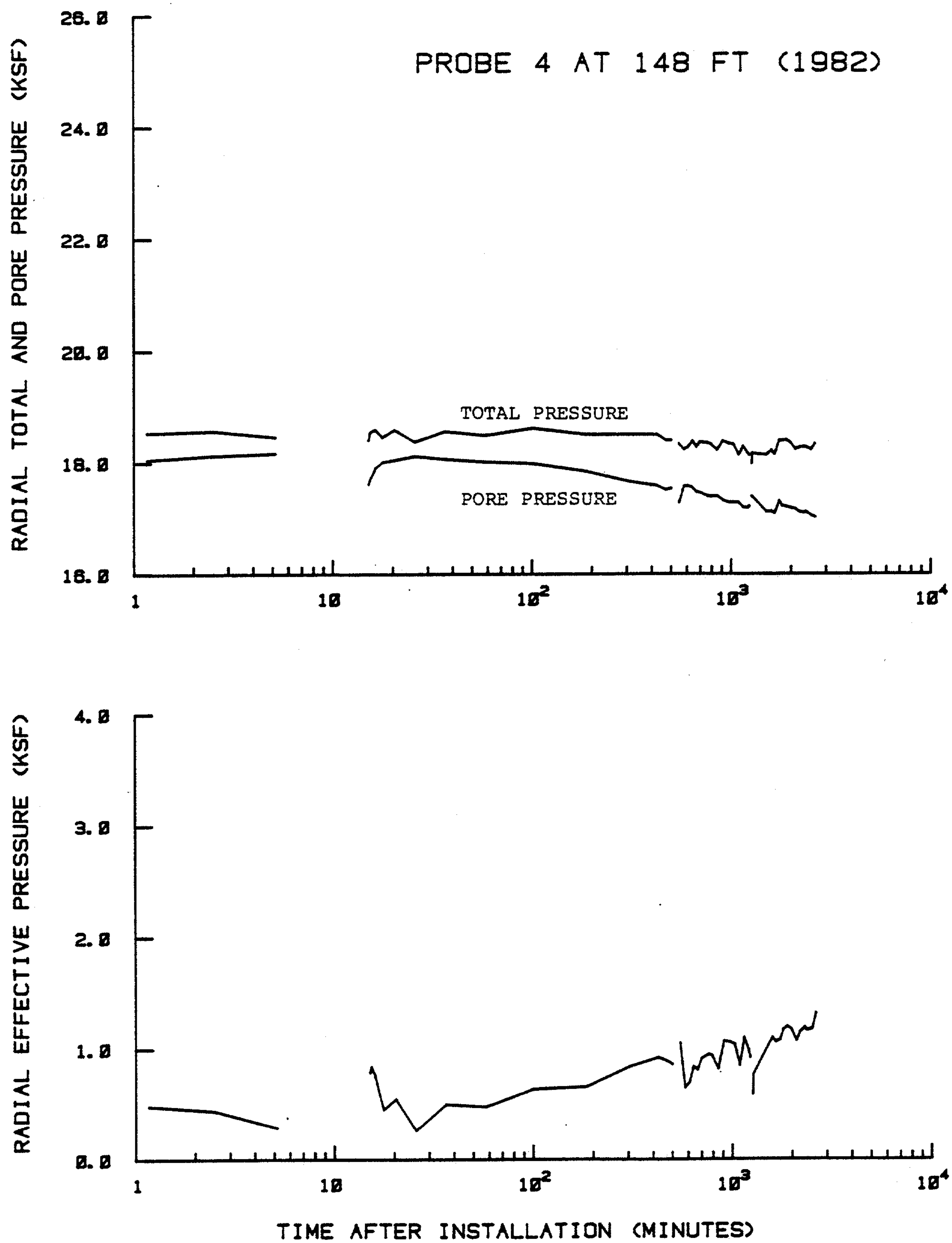


PROBE 3 AT 148 FT - PULL OUT TEST

EFFECTS OF LOAD RATE ON THE SOIL BEHAVIOR RECORDED

IN EXPERIMENT 2 AT 148 FEET

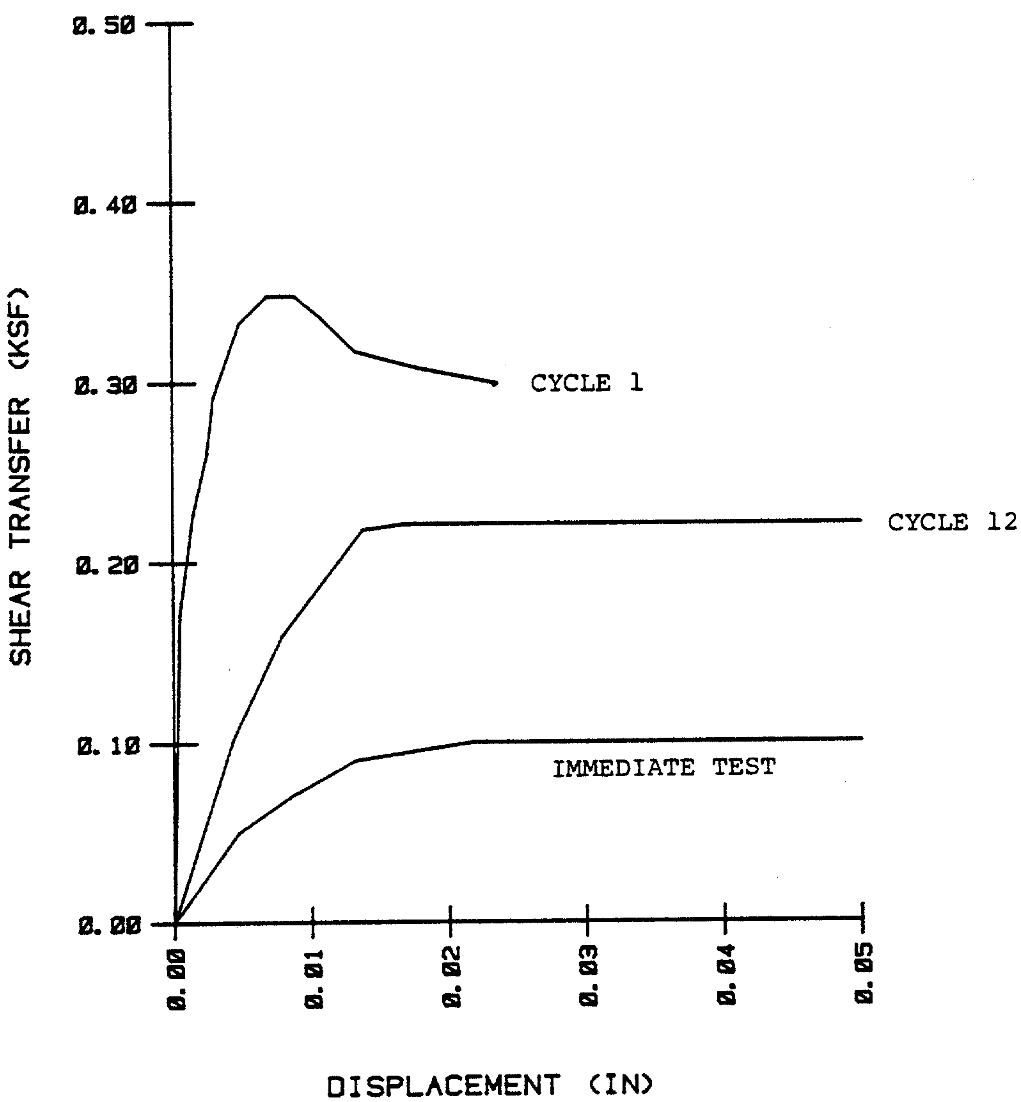
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 2 AT 148 FEET

PLATE 5.24

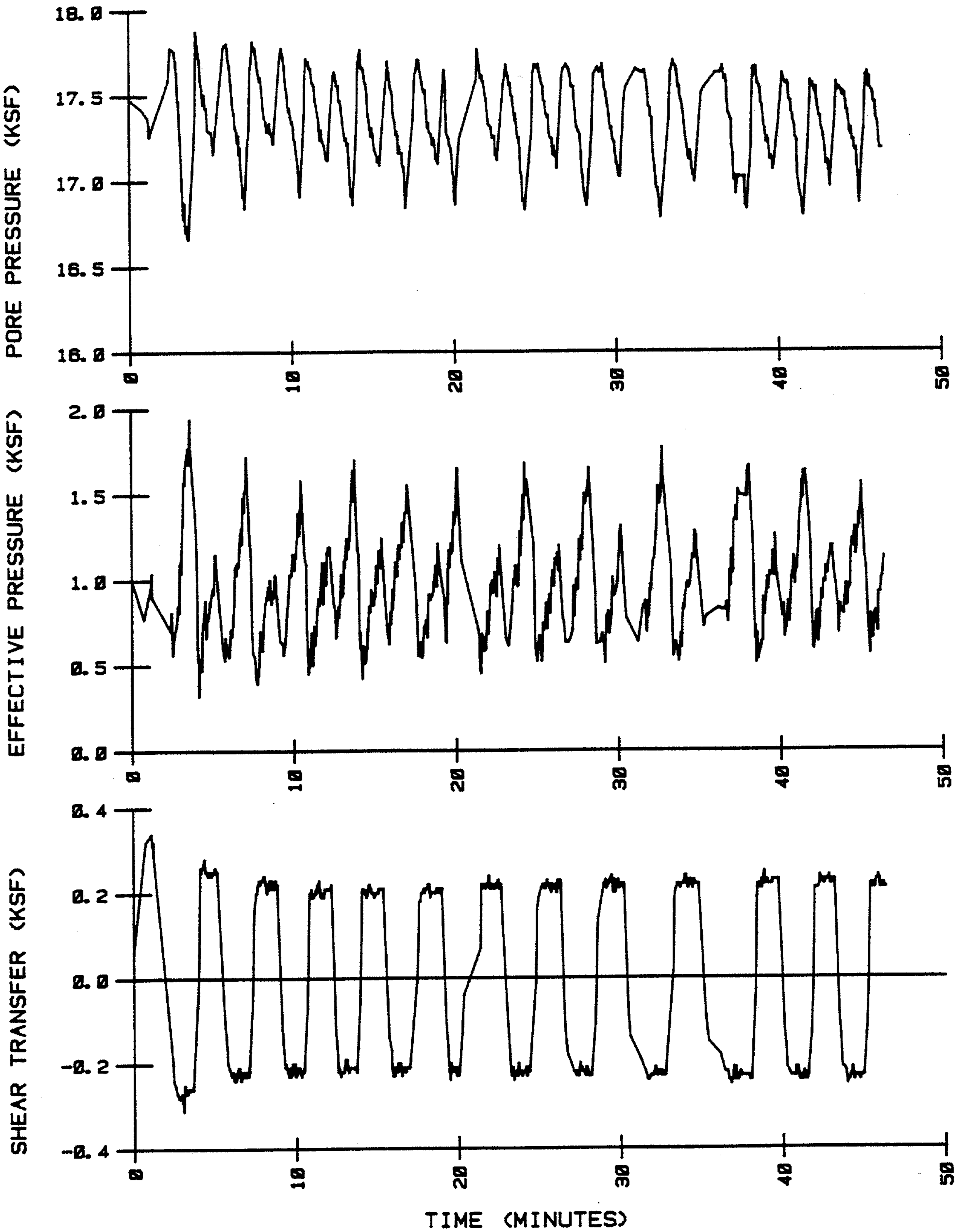
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 148 FT - 8 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 3 AT 148 FEET

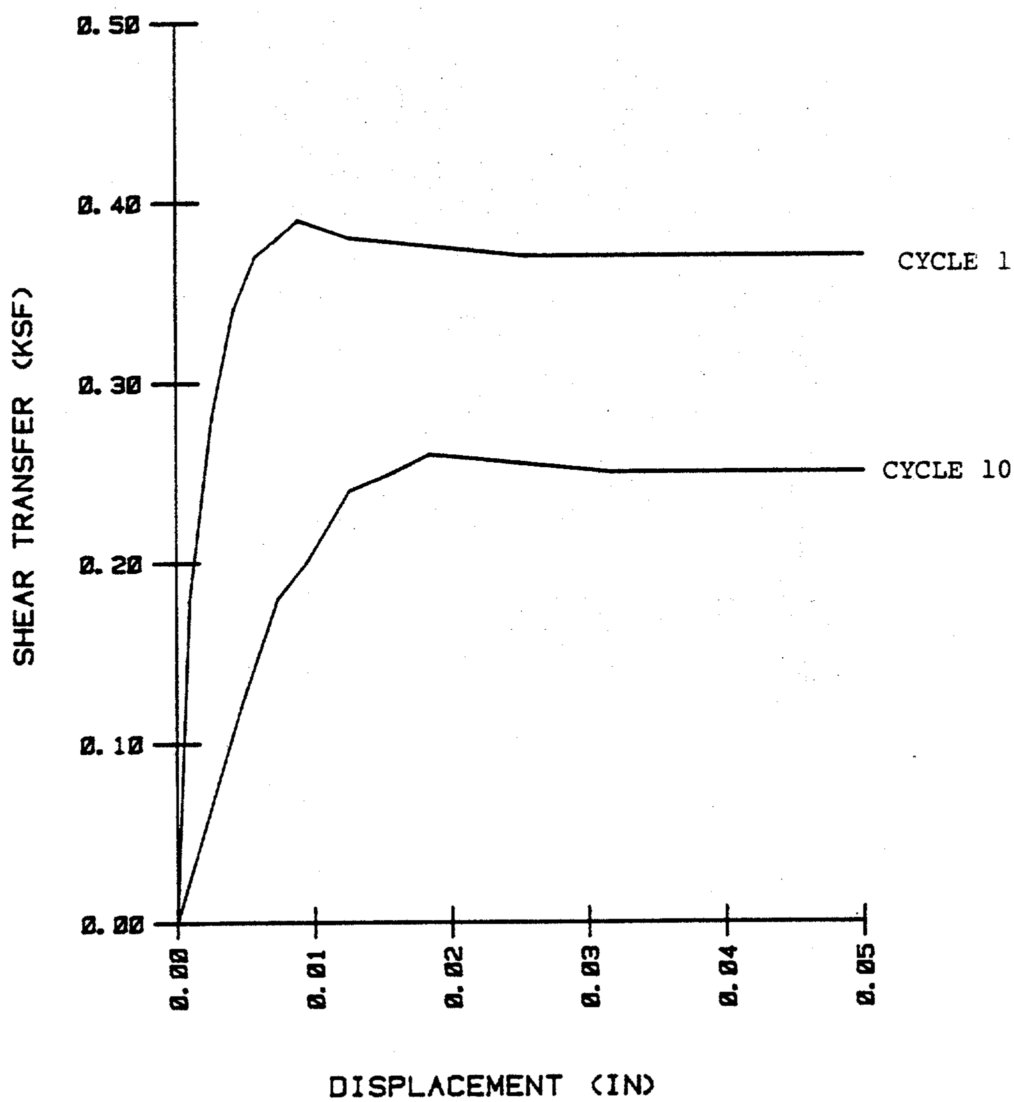
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 148 FT - 8 HR TEST

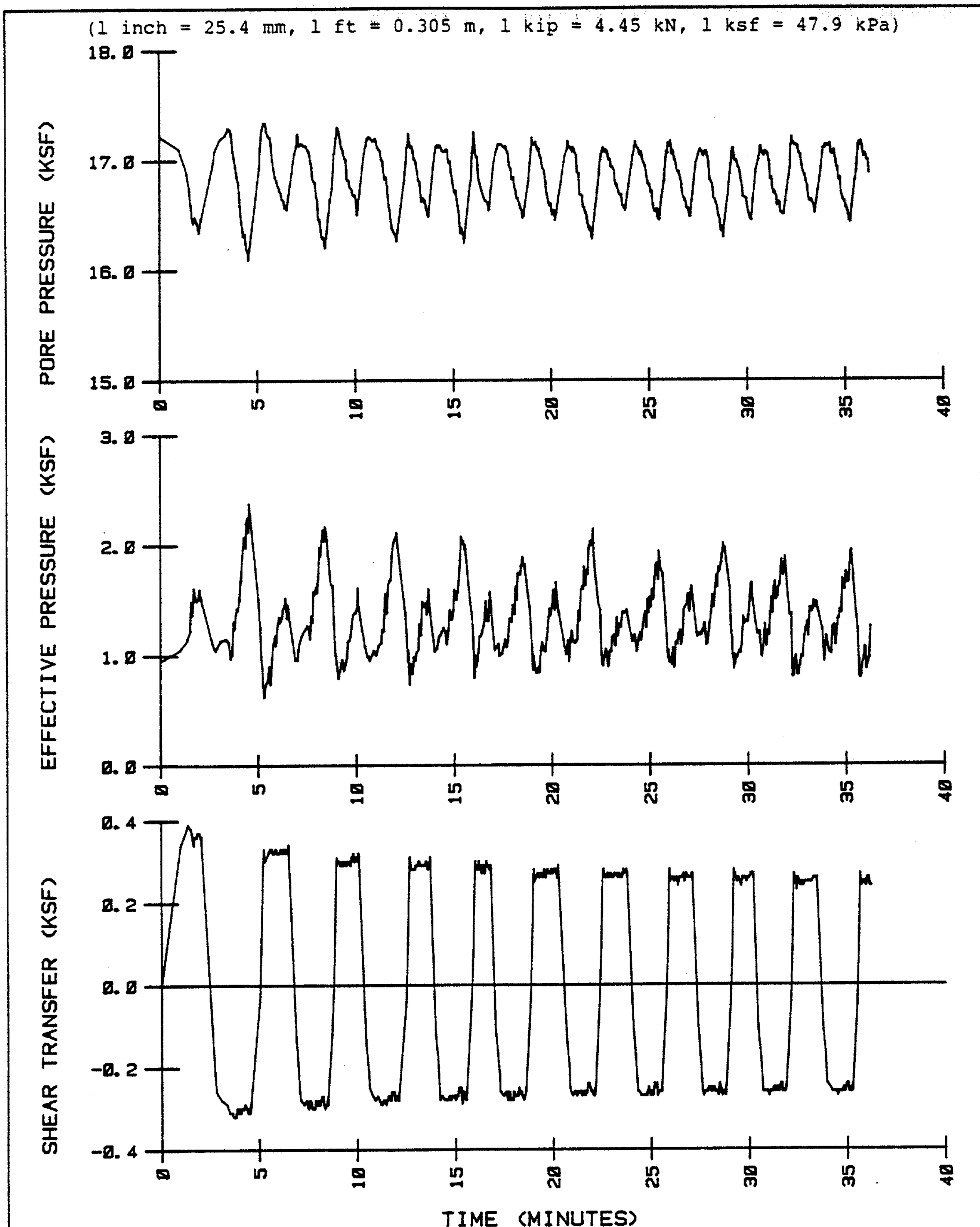
SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST
IN EXPERIMENT 3 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 148 FT - 20 HR TEST

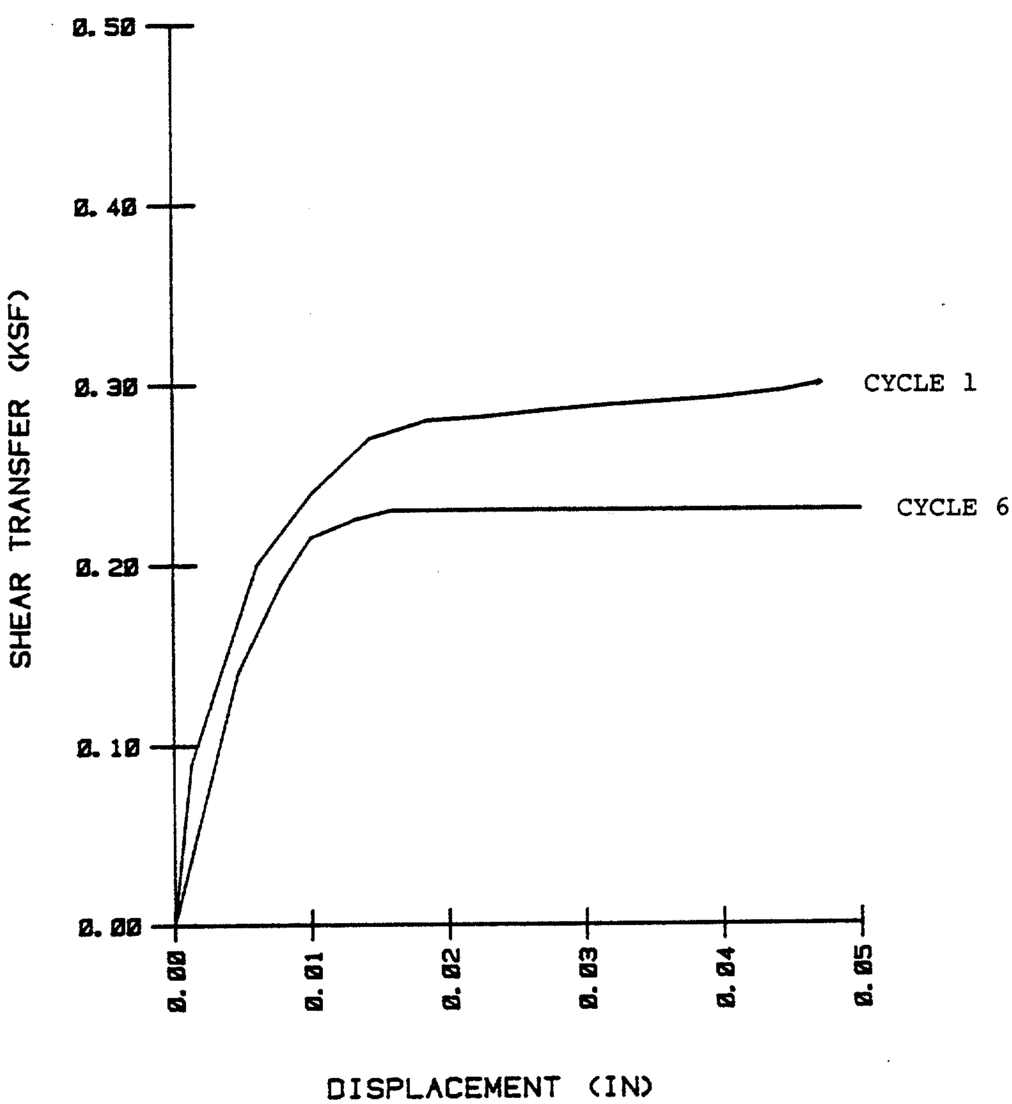
SHEAR TRANSFER CURVES RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 3 AT 148 FEET



PROBE 4 AT 148 FT - 20 HR TEST

SOIL BEHAVIOR RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 3 AT 148 FEET

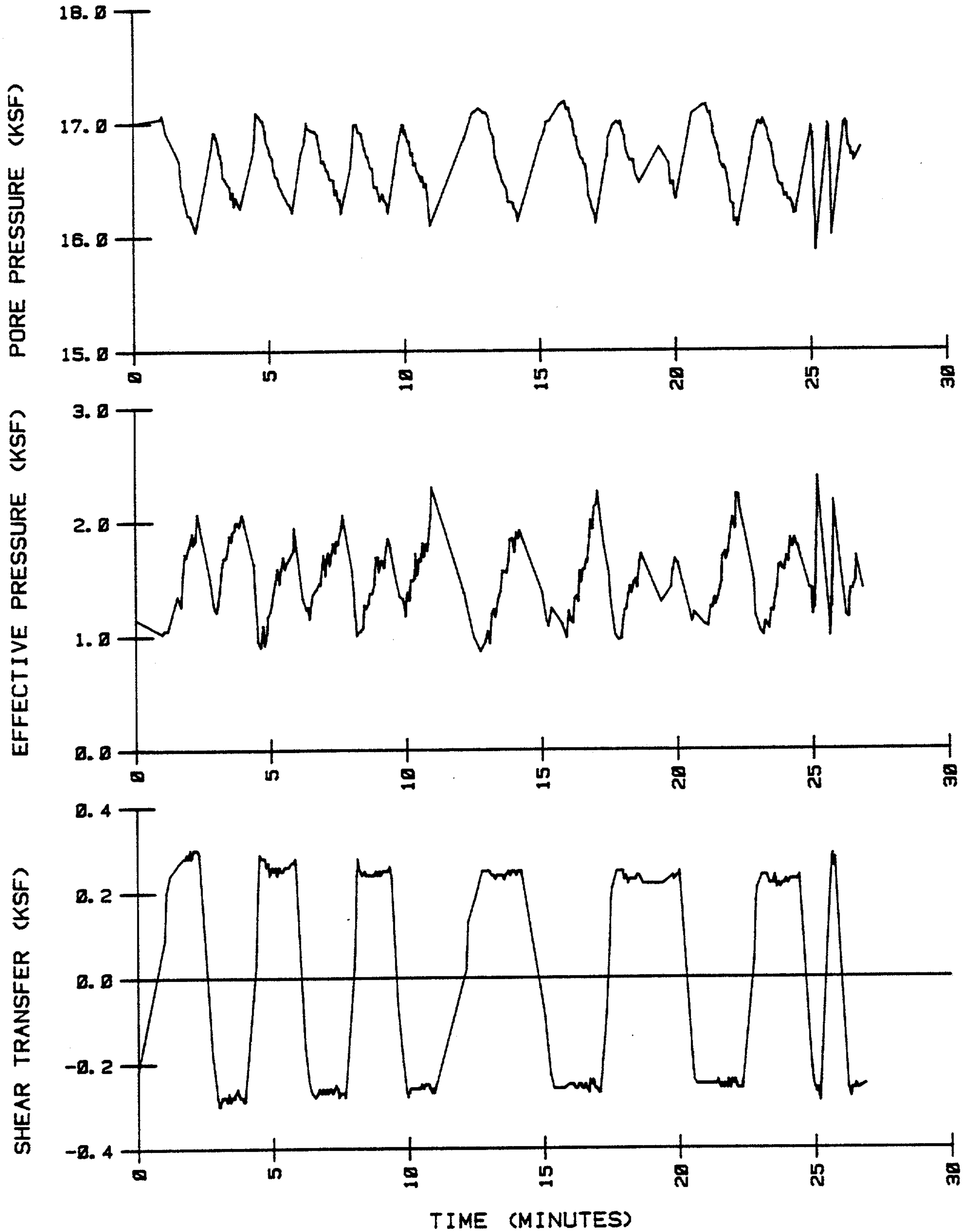
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 148 FT - 44 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE THIRD CYCLIC TEST IN EXPERIMENT 3 AT 148 FEET

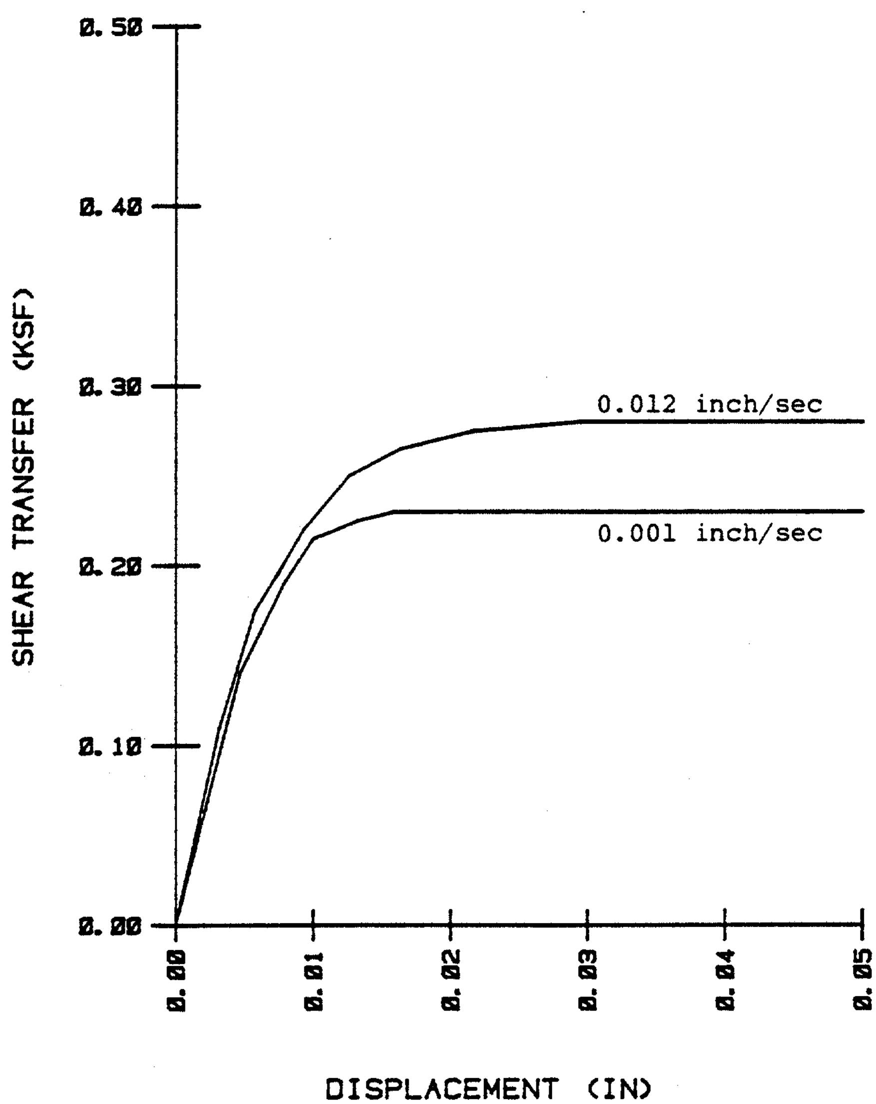
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 148 FT - 44 HR TEST

SOIL BEHAVIOR RECORDED DURING THE THIRD CYCLIC TEST IN EXPERIMENT 3 AT 148 FEET

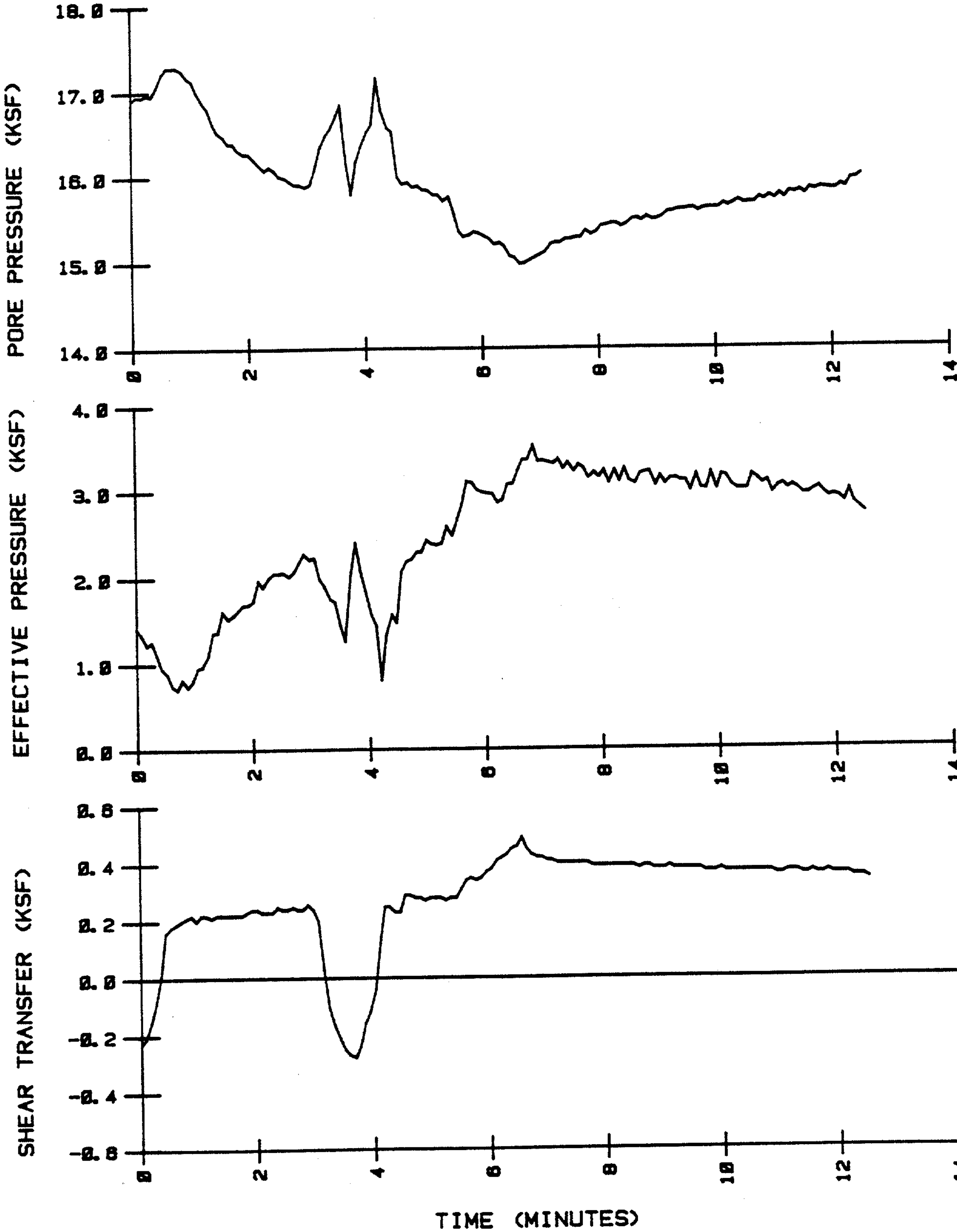
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 148 FT - RATE STUDY

EFFECTS OF LOAD RATE ON THE SHEAR TRANSFER RECORDED IN EXPERIMENT 3 AT 148 FEET

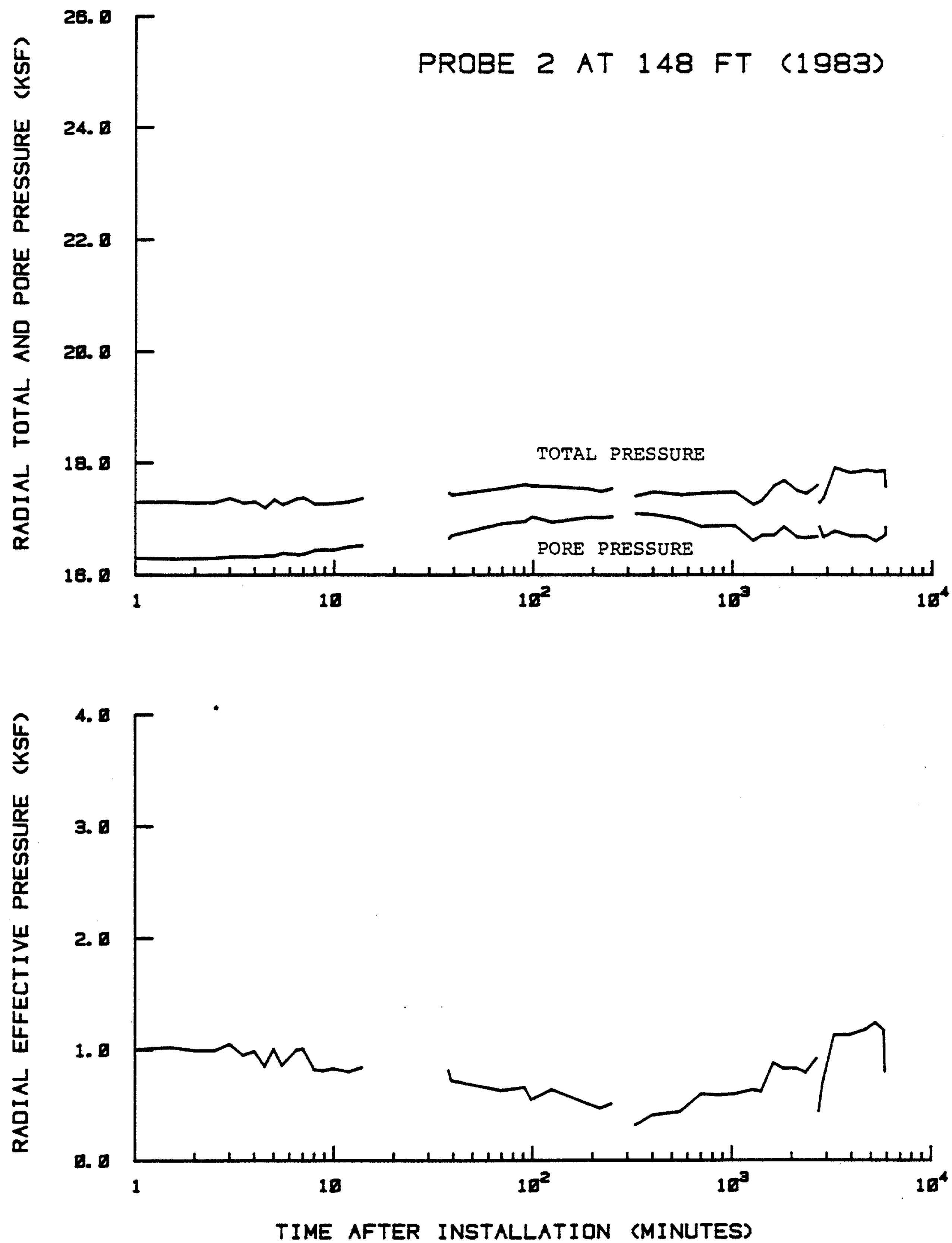
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 148 FT - PULL OUT TEST

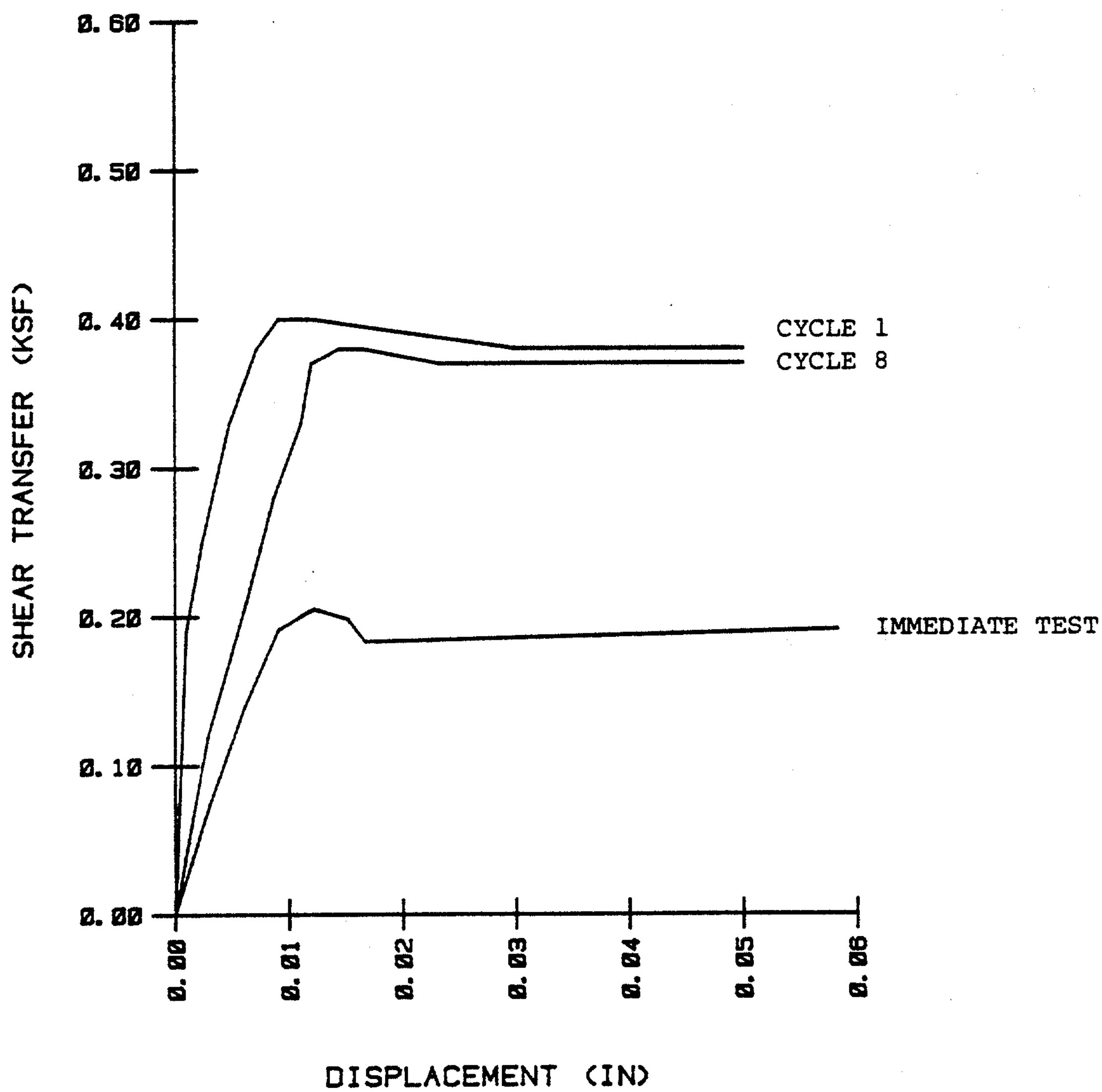
EFFECTS OF LOAD RATE ON THE SOIL BEHAVIOR RECORDED IN EXPERIMENT 3 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



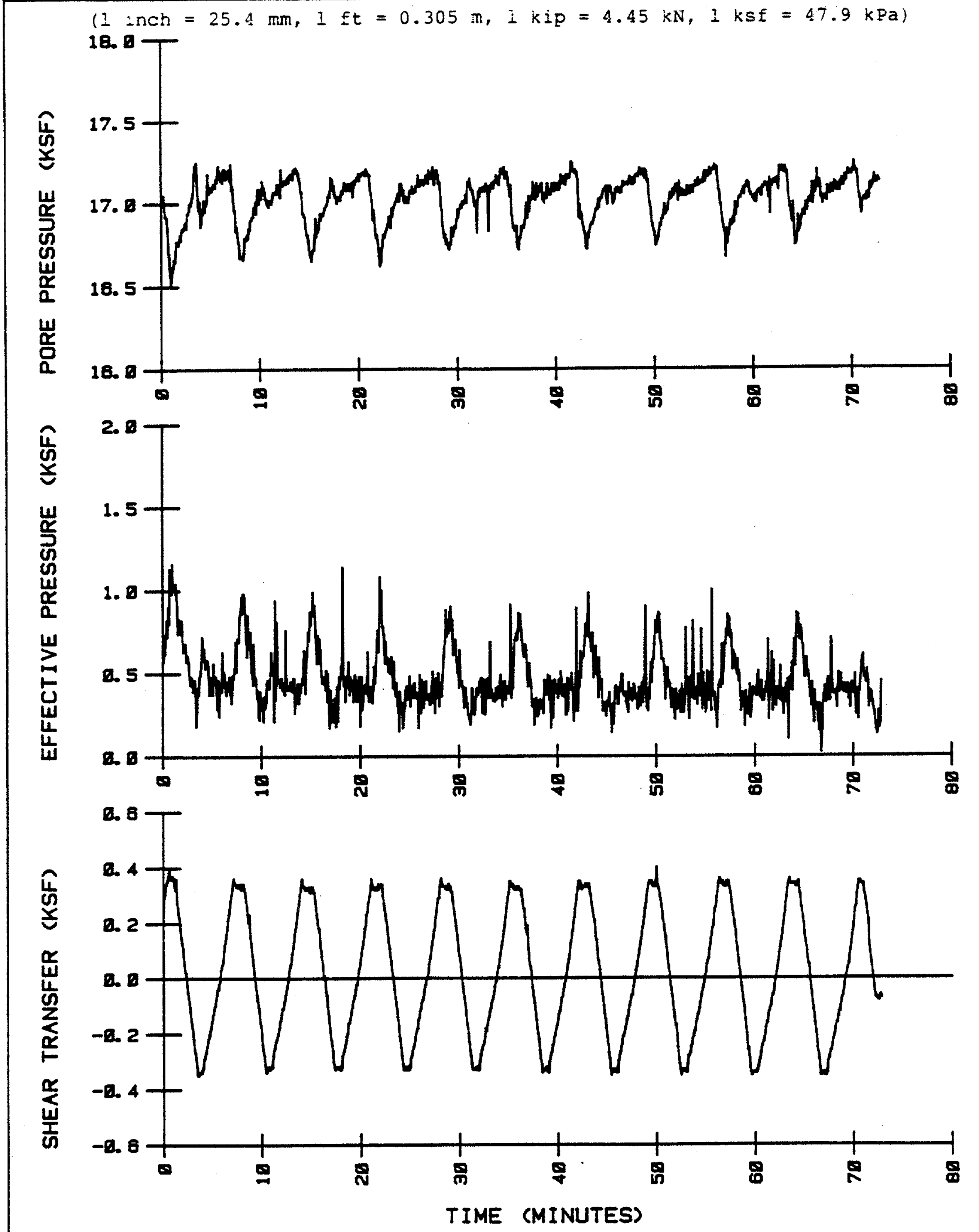
SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 4 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 2 AT 148 FT - 4 HR TEST

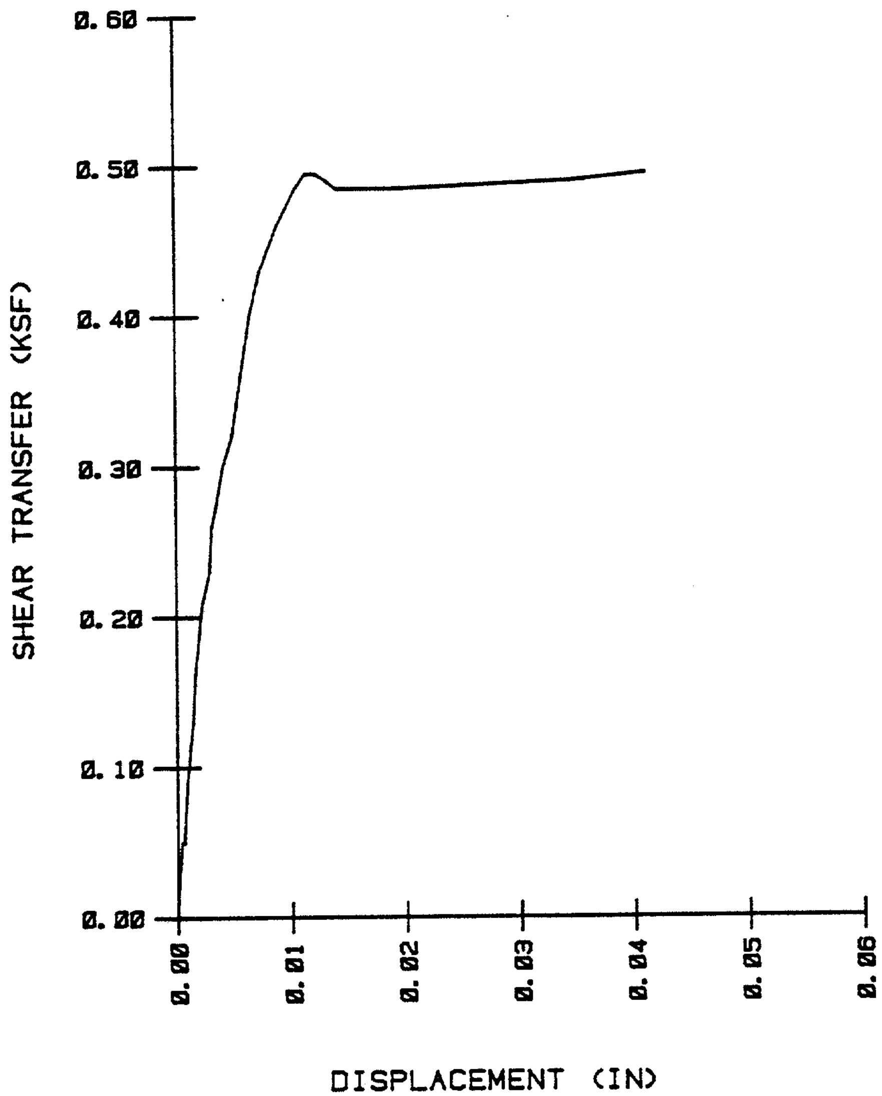
SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 4 AT 148 FEET



PROBE 2 AT 148 FT - 4 HR TEST

SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 4 AT 148 FEET

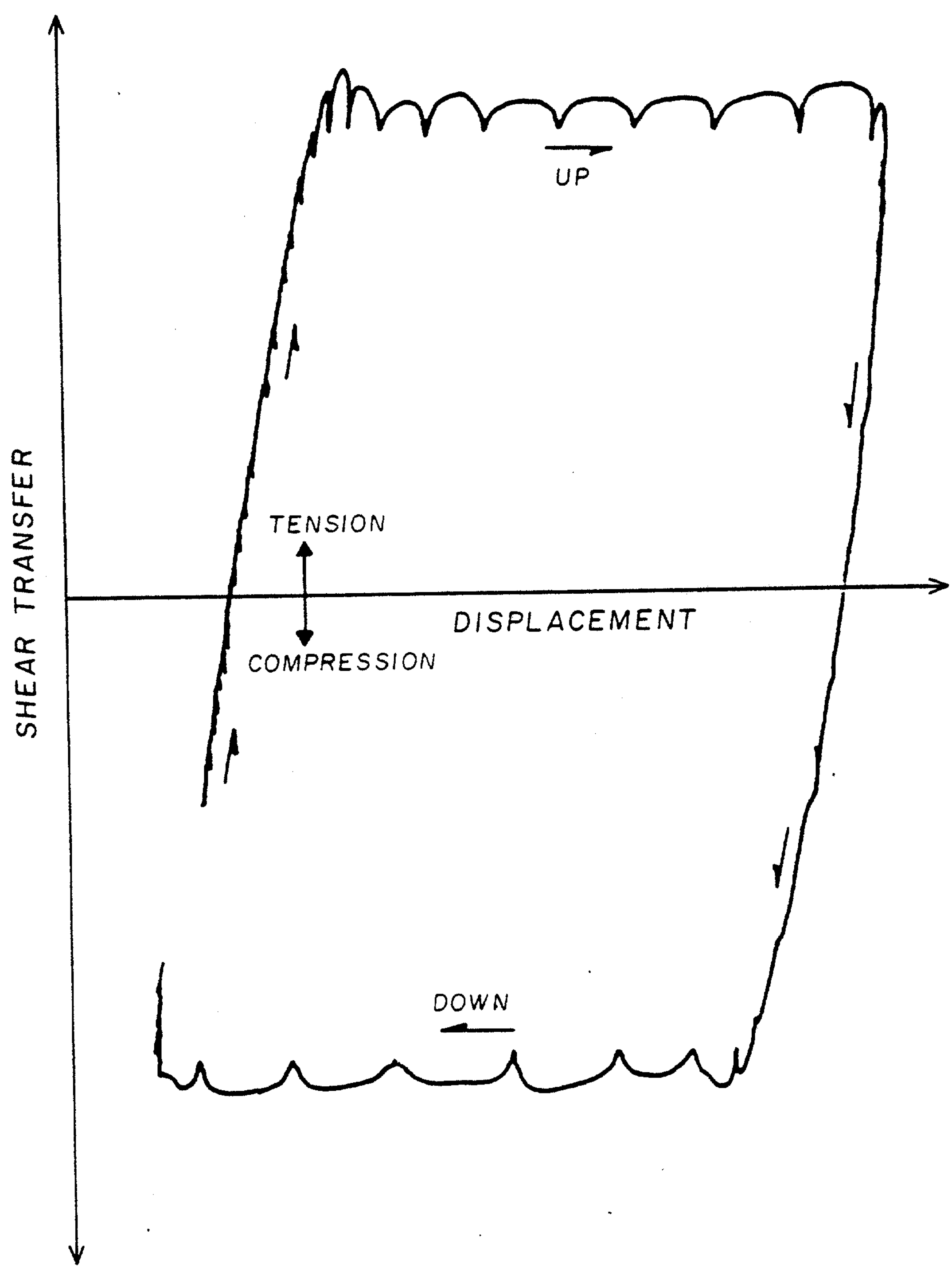
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 2 AT 148 FT - 44 HR TEST

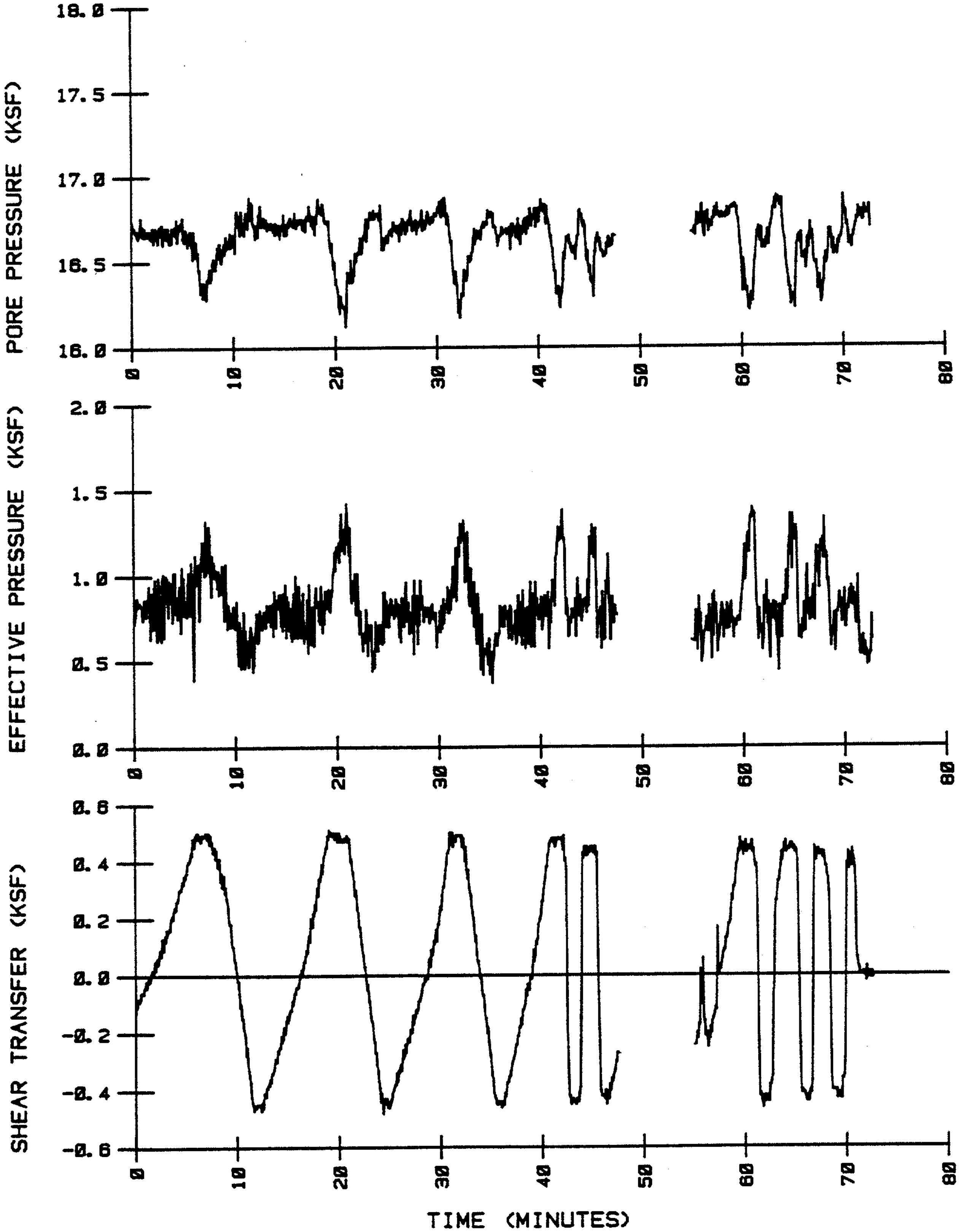
SHEAR TRANSFER CURVE RECORDED DURING THE FIRST CYCLE OF THE SECOND CYCLIC TEST IN EXPERIMENT 4 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



COMPLETE SHEAR TRANSFER CURVE RECORDED DURING THE SIXTH CYCLE OF THE SECOND CYCLIC TEST IN EXPERIMENT 4 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)

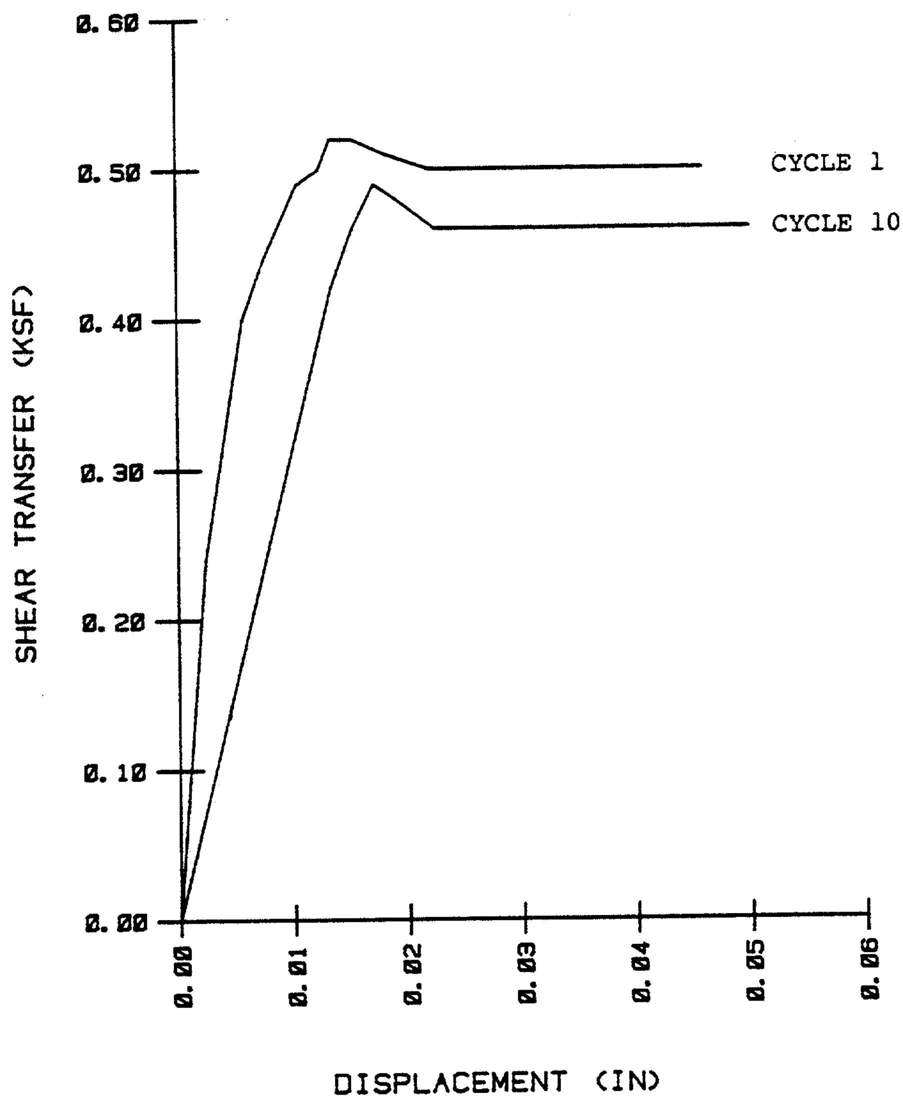


PROBE 2 AT 148 FT - 44 HR TEST

SOIL BEHAVIOR RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 4 AT 148 FEET

PLATE 5.38

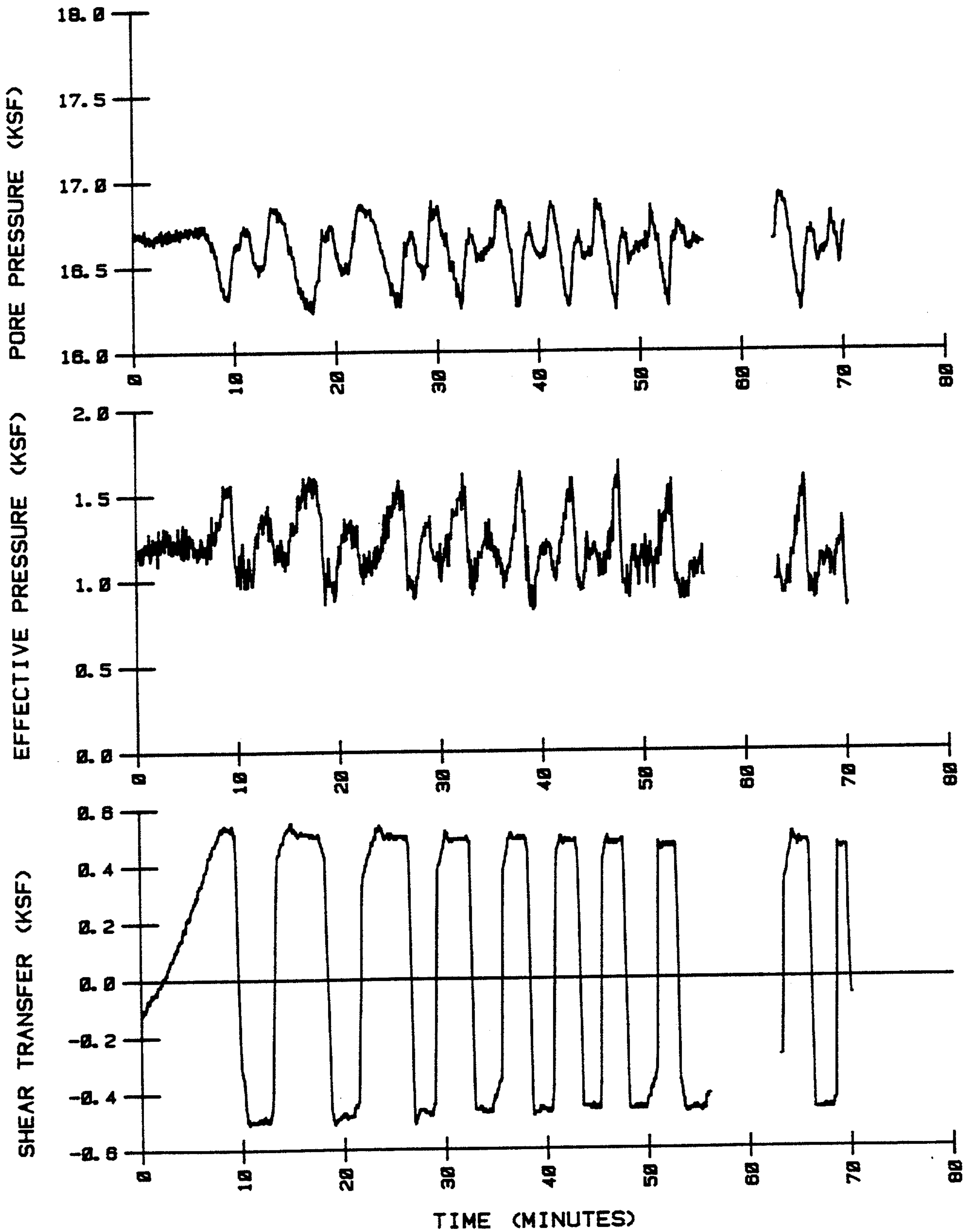
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 2 AT 148 FT - 96 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE THIRD CYCLIC TEST IN EXPERIMENT 4 AT 148 FEET

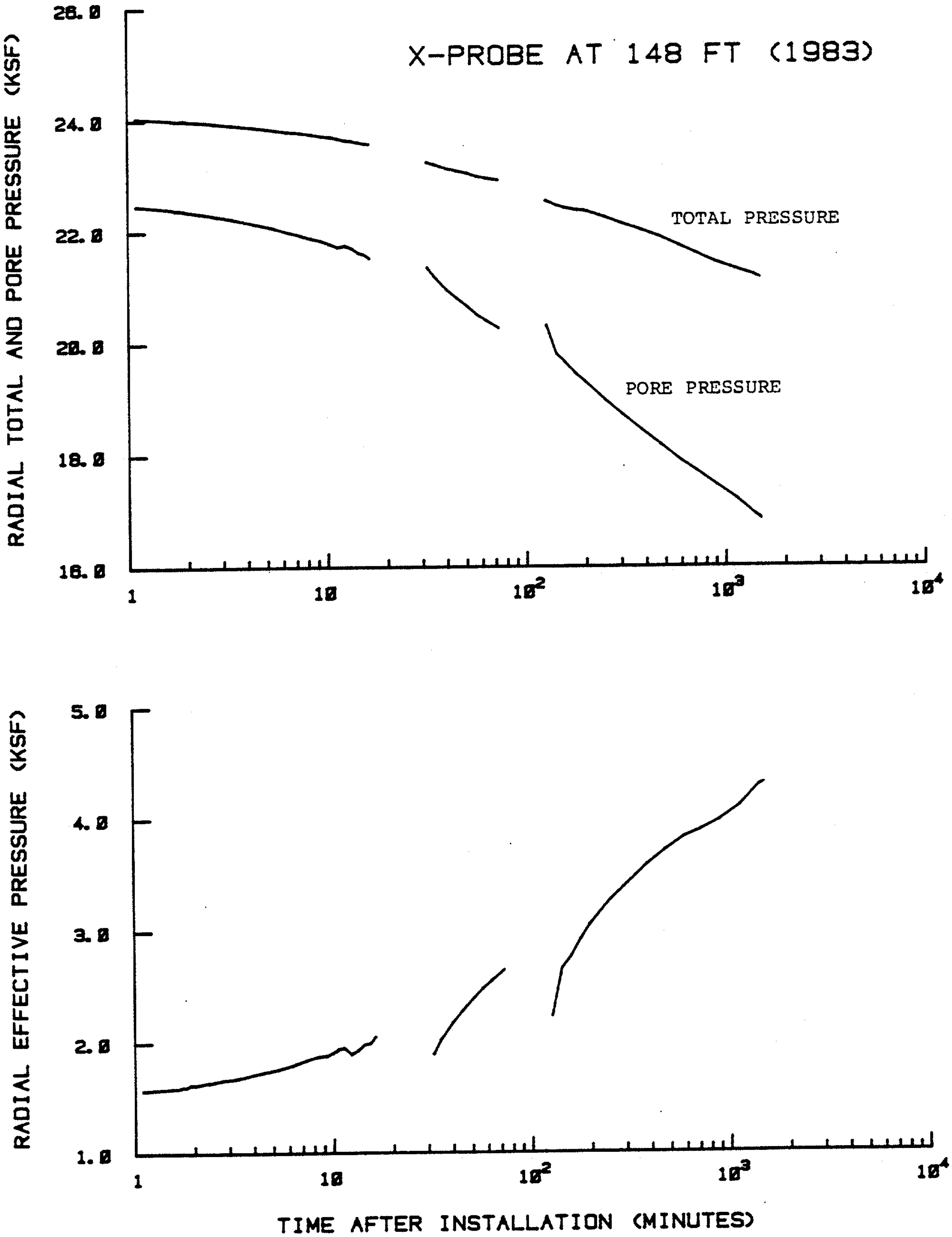
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 2 AT 148 FT - 96 HR TEST

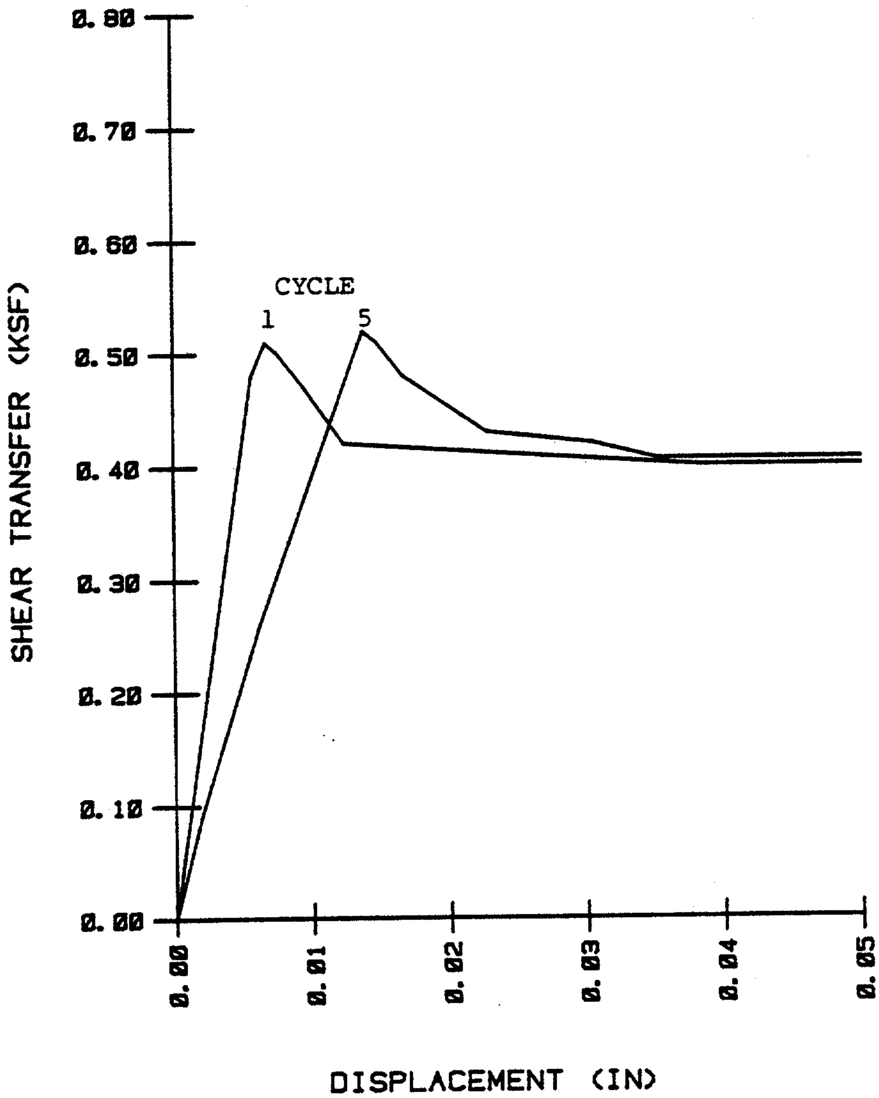
SOIL BEHAVIOR RECORDED DURING THE THIRD CYCLIC TEST IN EXPERIMENT 4 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 5 AT 148 FEET

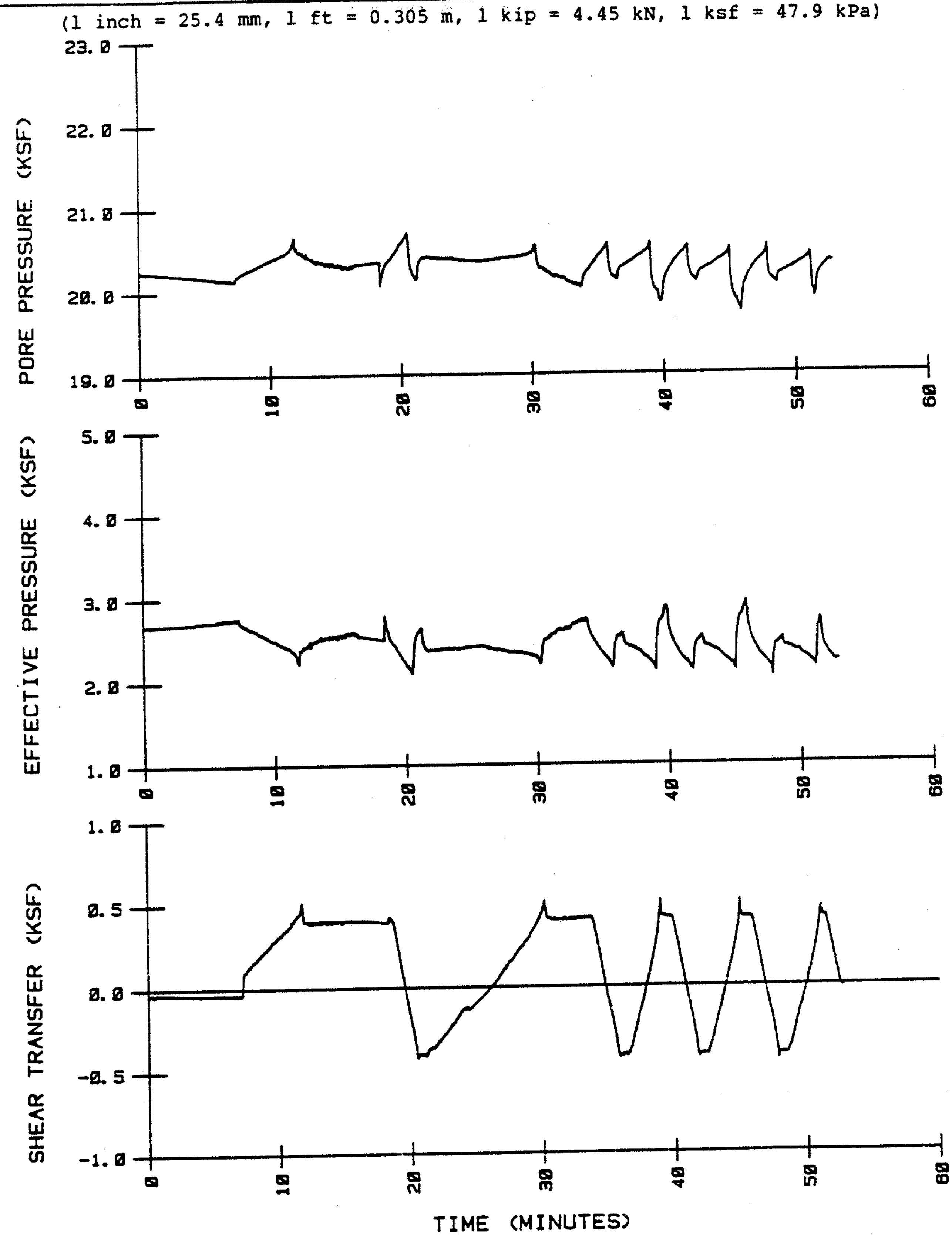
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



X-PROBE AT 148 FT - 1 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 5 AT 148 FEET

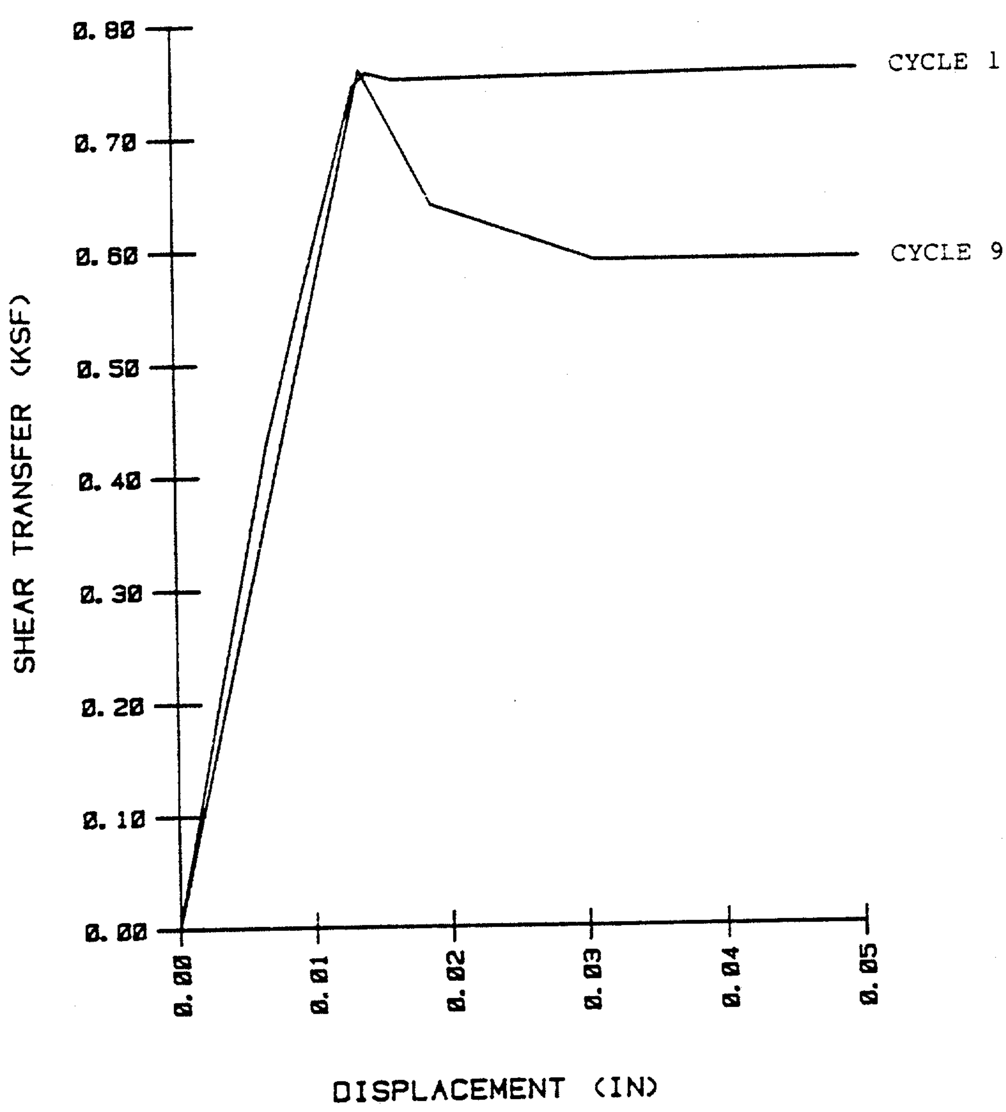
PLATE 5.42



X-PROBE AT 148 FT - 1 HR TEST

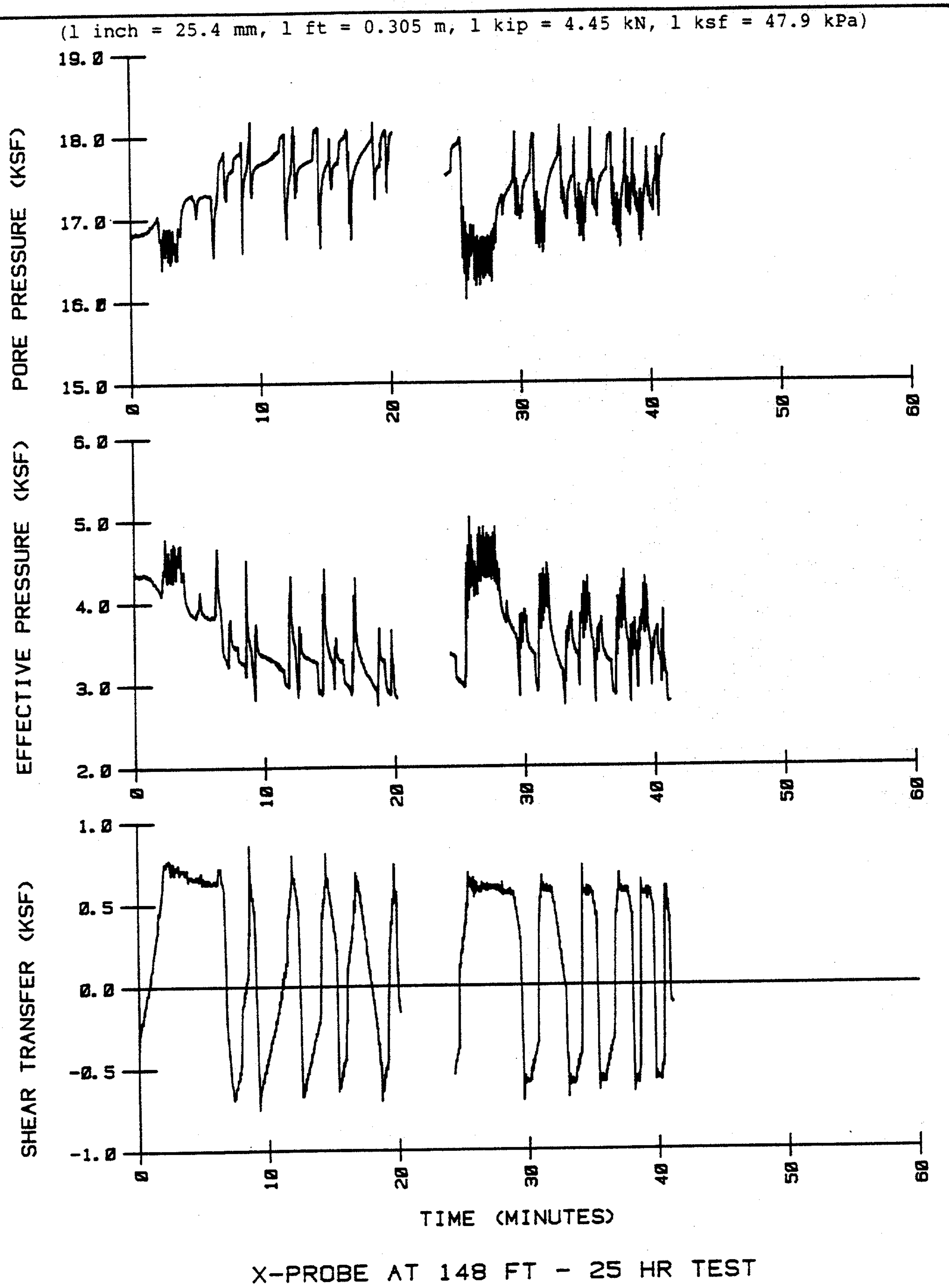
SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 5 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



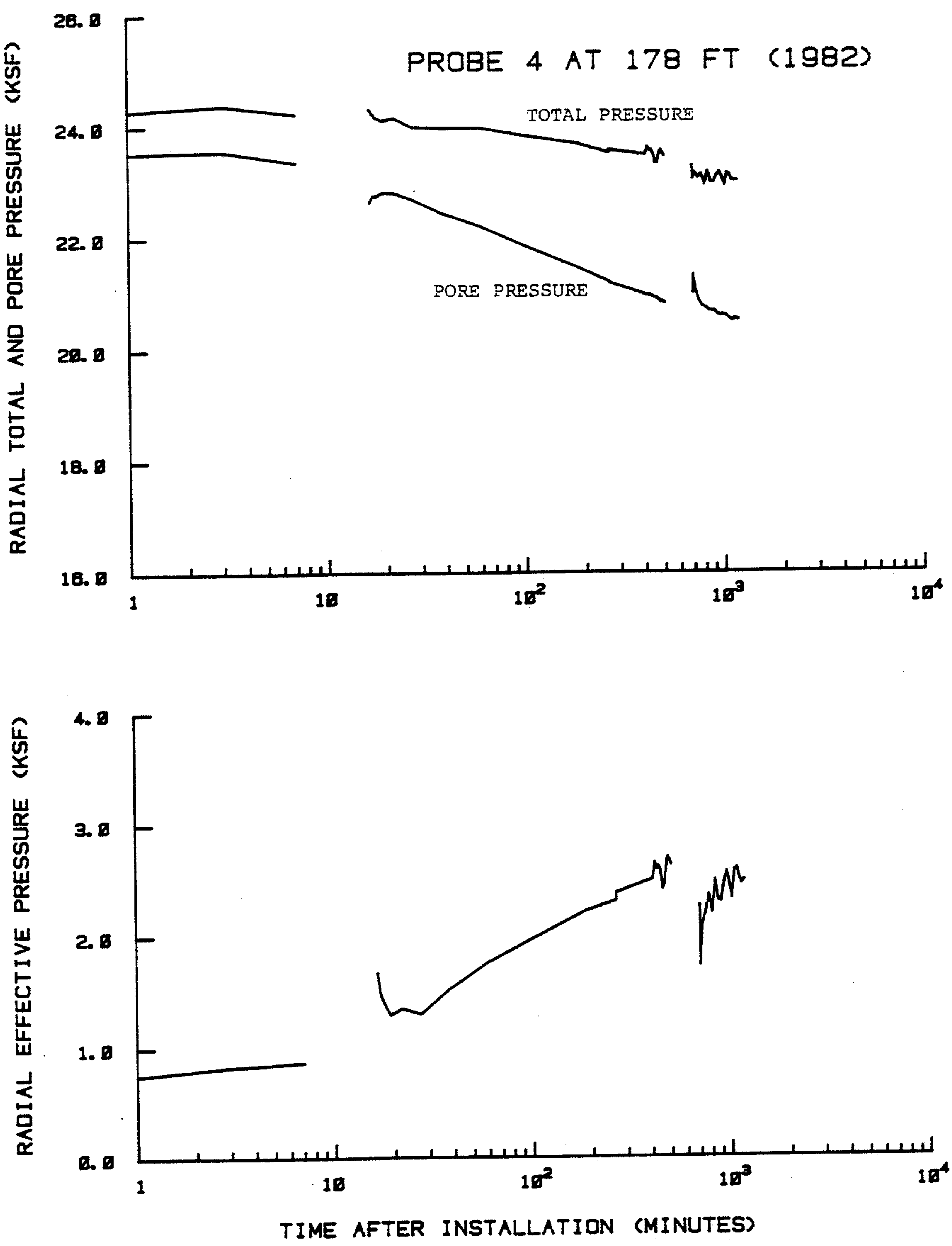
X-PROBE AT 148 FT - 25 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 5 AT 148 FEET



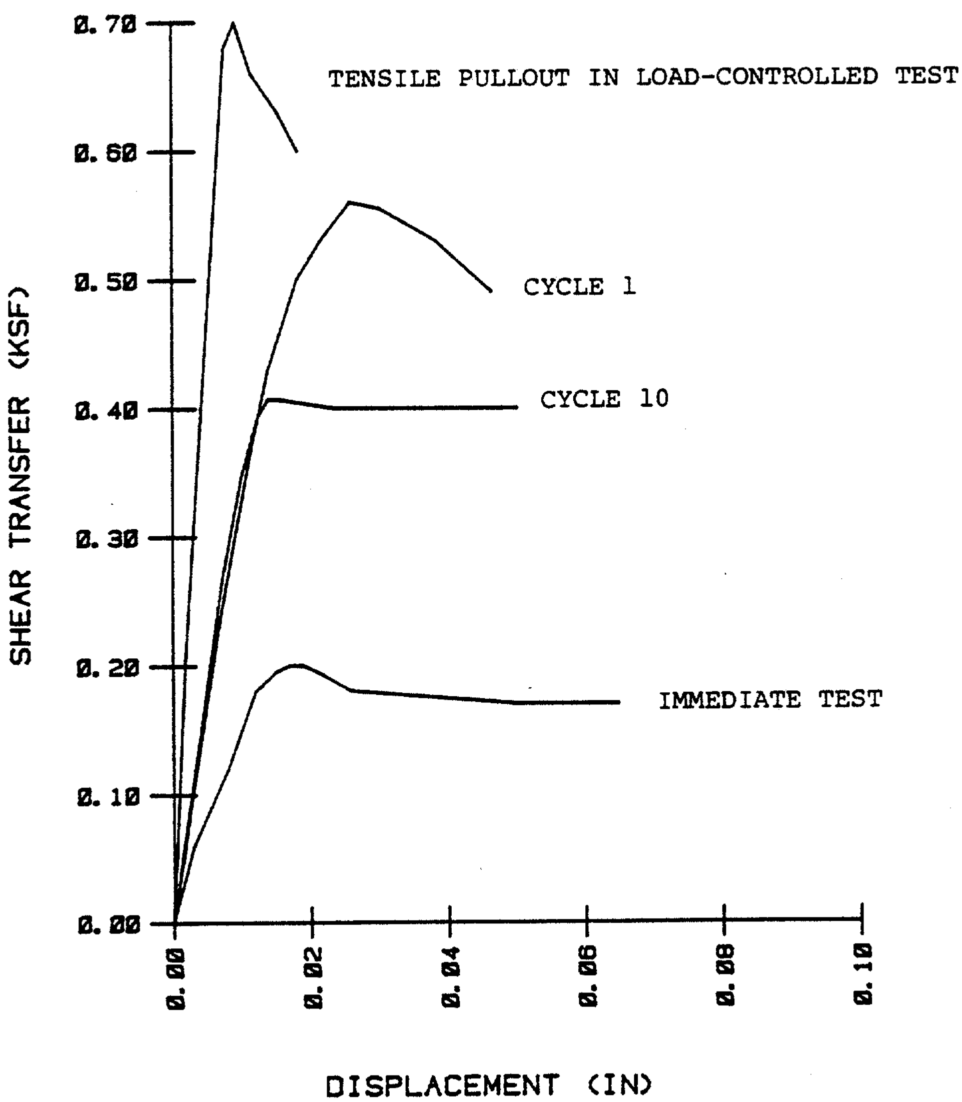
SOIL BEHAVIOR RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 5 AT 148 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 1 AT 178 FEET

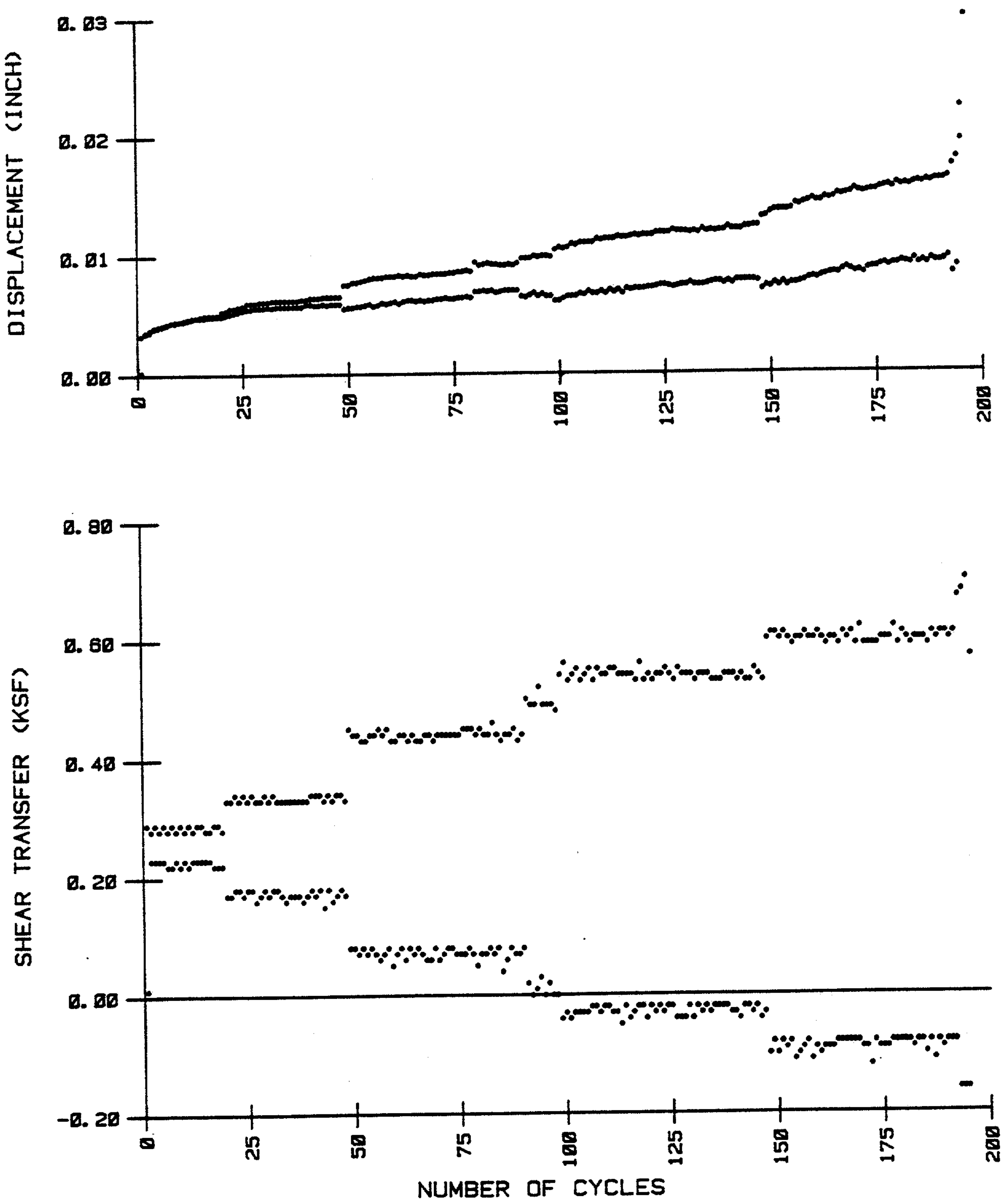
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 178 FT - 8 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 1 AT 178 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)

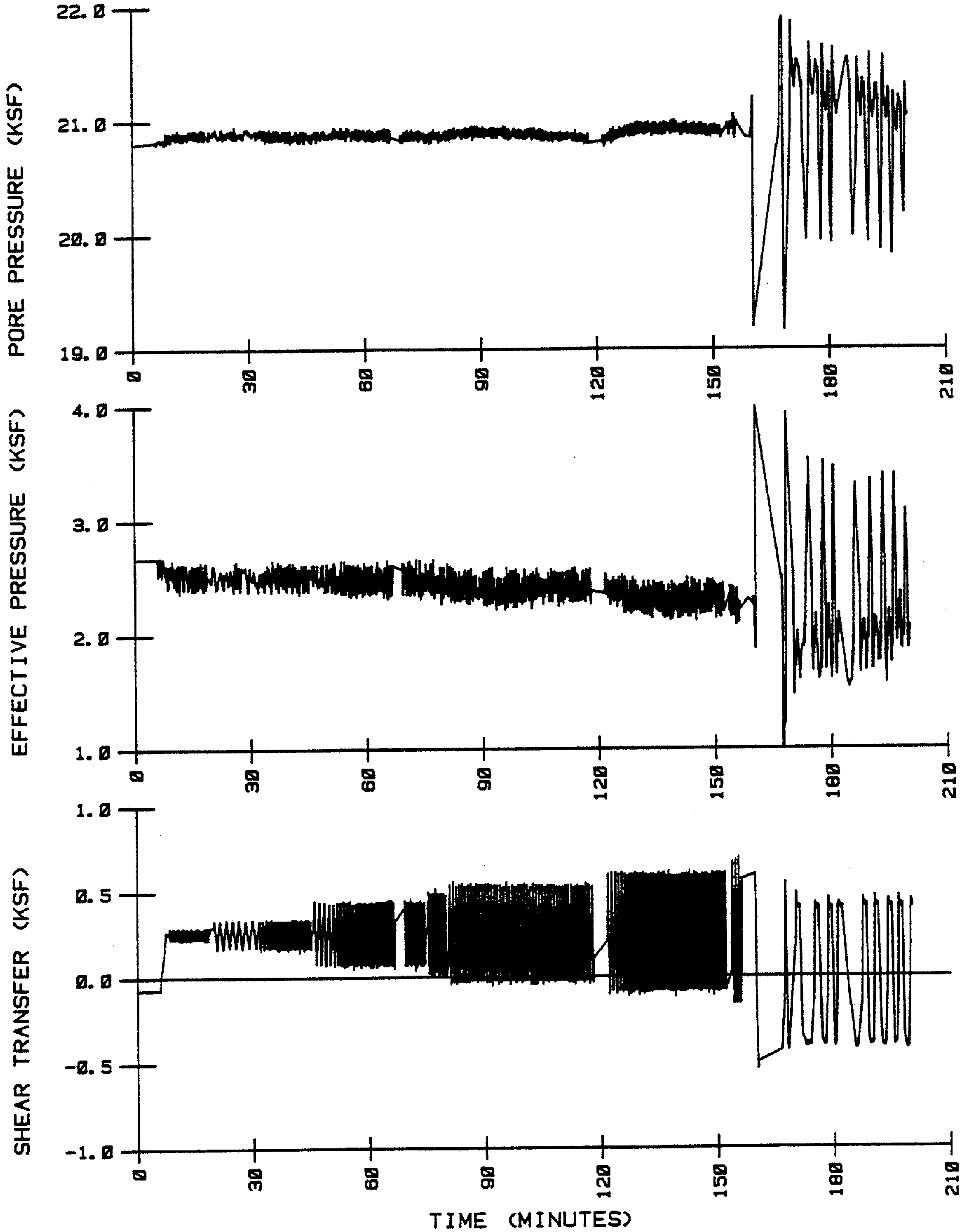


LOAD CONTROLLED CYCLIC TEST

VARIATION IN SHEAR TRANSFER AND DISPLACEMENT DURING LOAD-CONTROLLED CYCLIC TEST IN EXPERIMENT 1 AT 178 FEET

PLATE 5.48

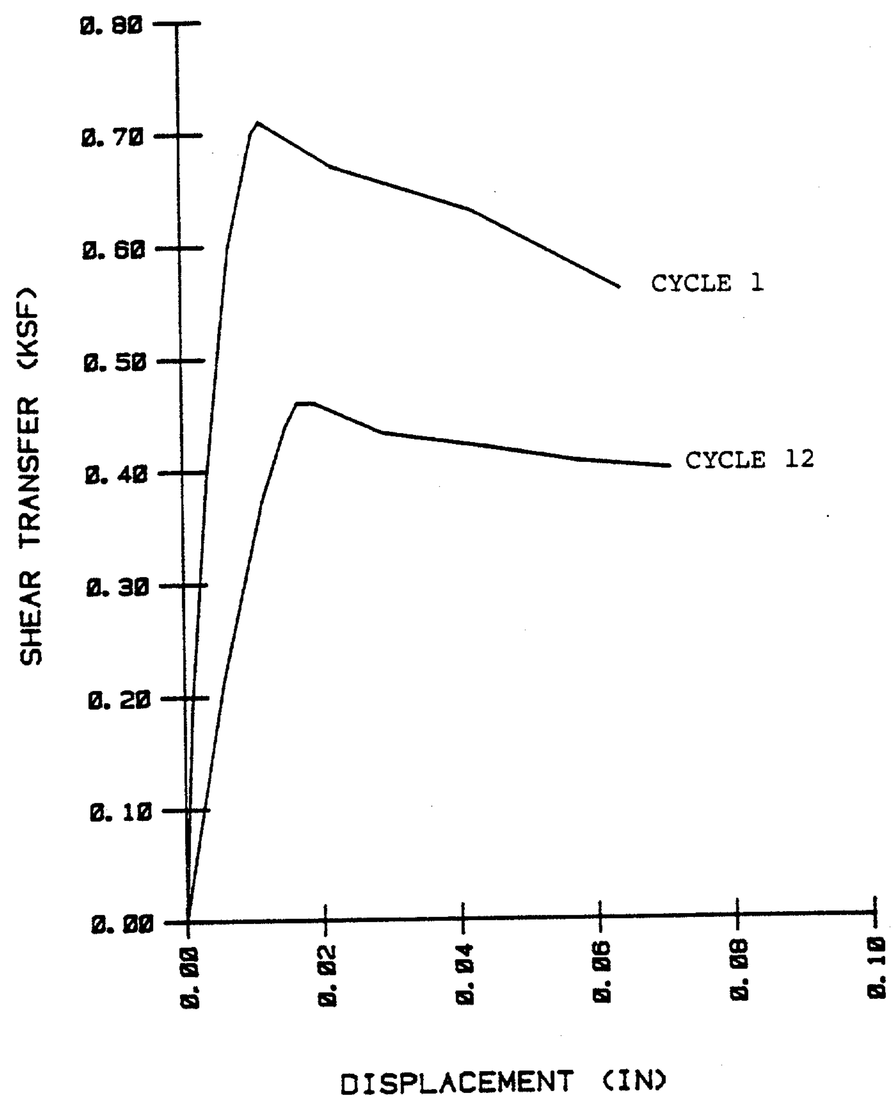
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 178 FT - 8 HR TEST

SOIL BEHAVIOR RECORDED DURING THE LOAD-CONTROLLED CYCLIC TEST IN EXPERIMENT 1 AT 178 FEET

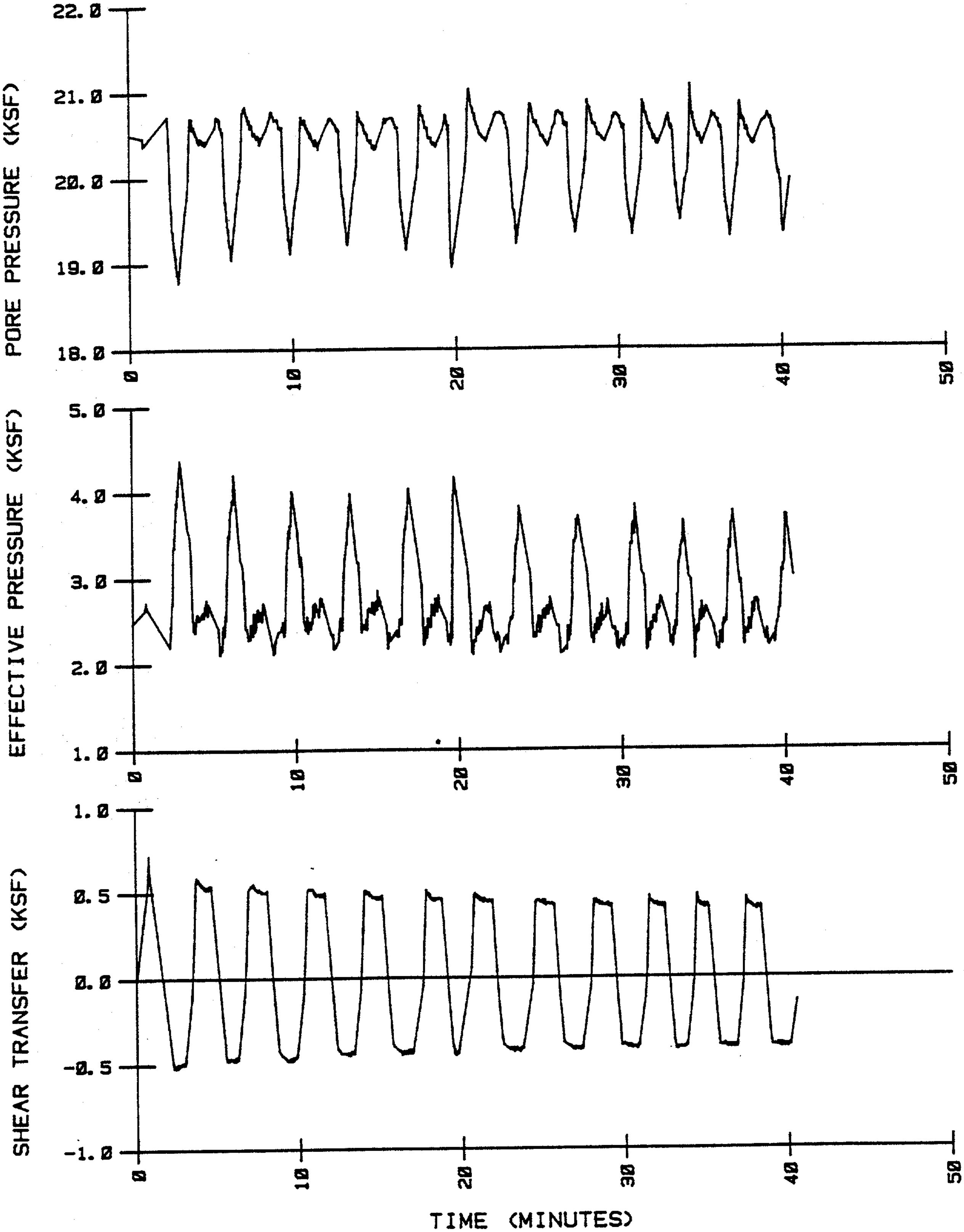
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 178 FT - 20 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 1 AT 178 FEET

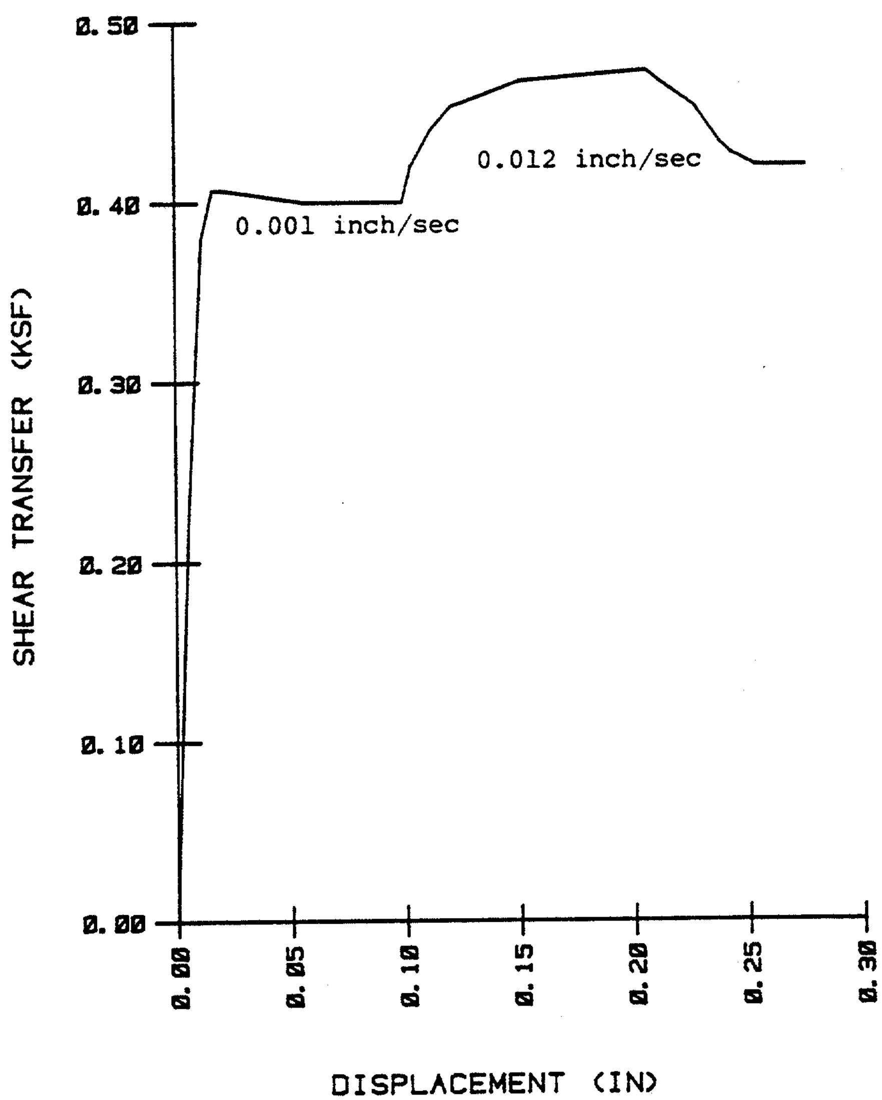
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 178 FT - 20 HR TEST

SOIL BEHAVIOR RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 1 AT 178 FEET

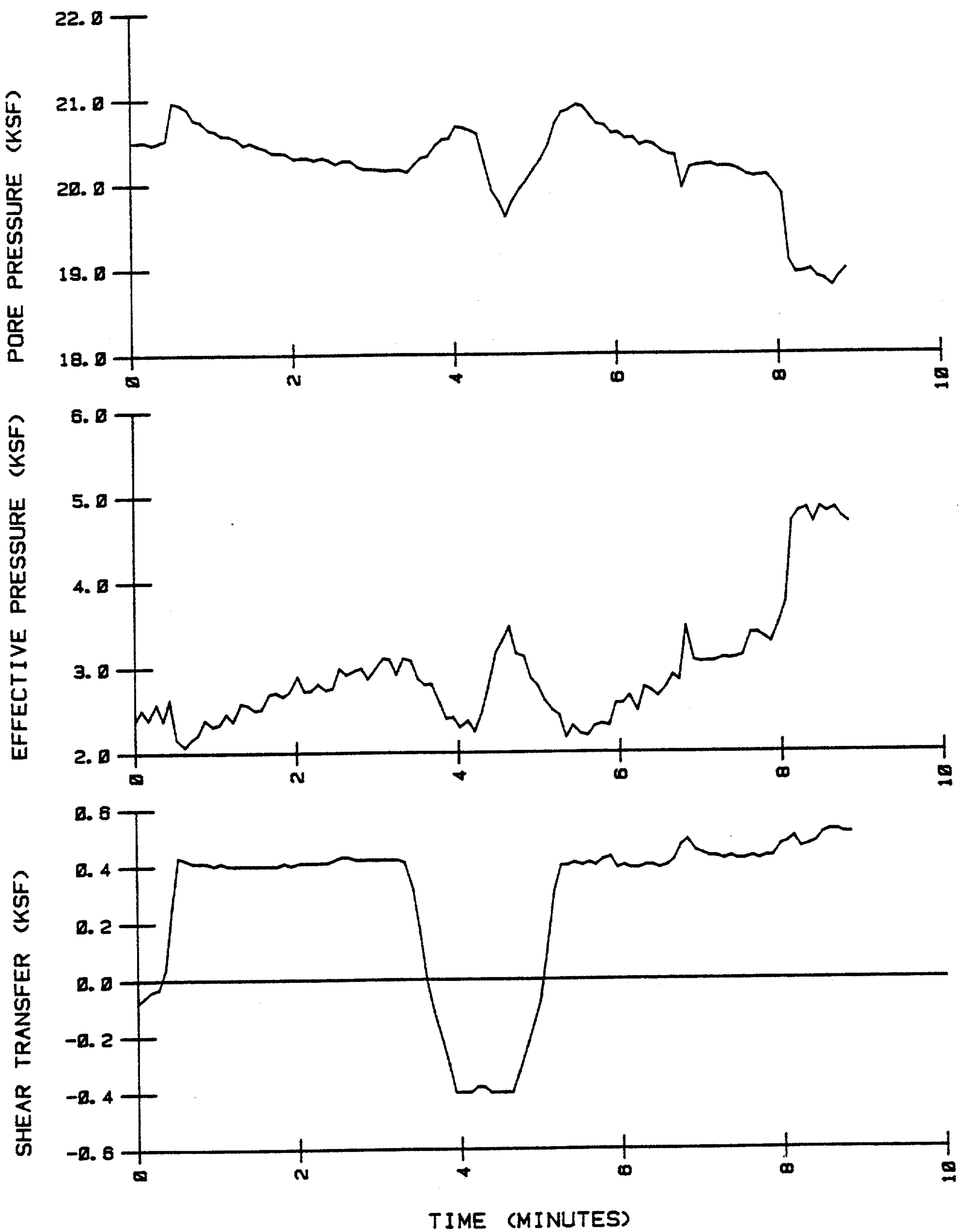
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 178 FT - PULL OUT TEST

EFFECTS OF LOAD RATE ON THE SHEAR TRANSFER RECORDED IN EXPERIMENT 1 AT 178 FEET

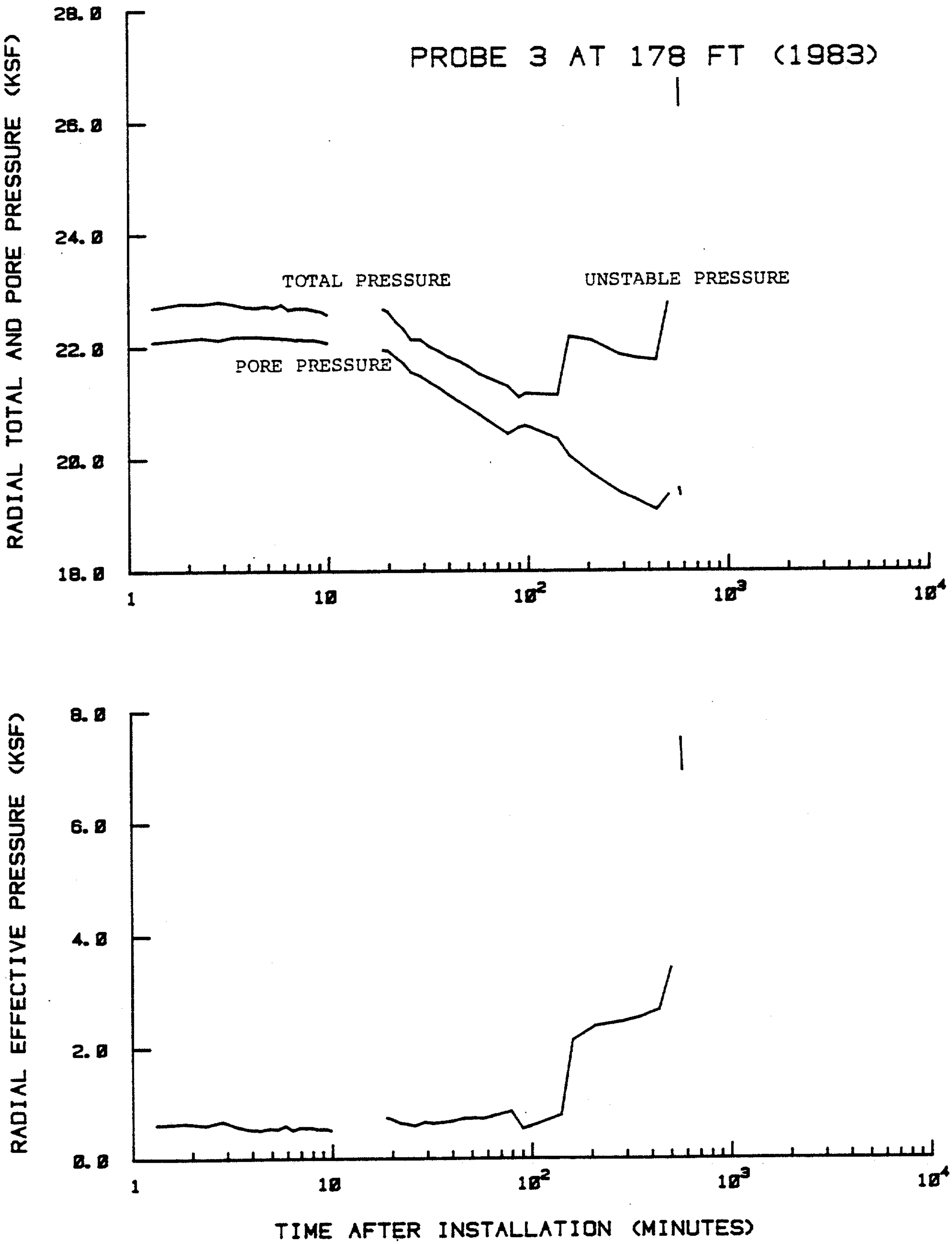
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 178 FT - PULL OUT TEST

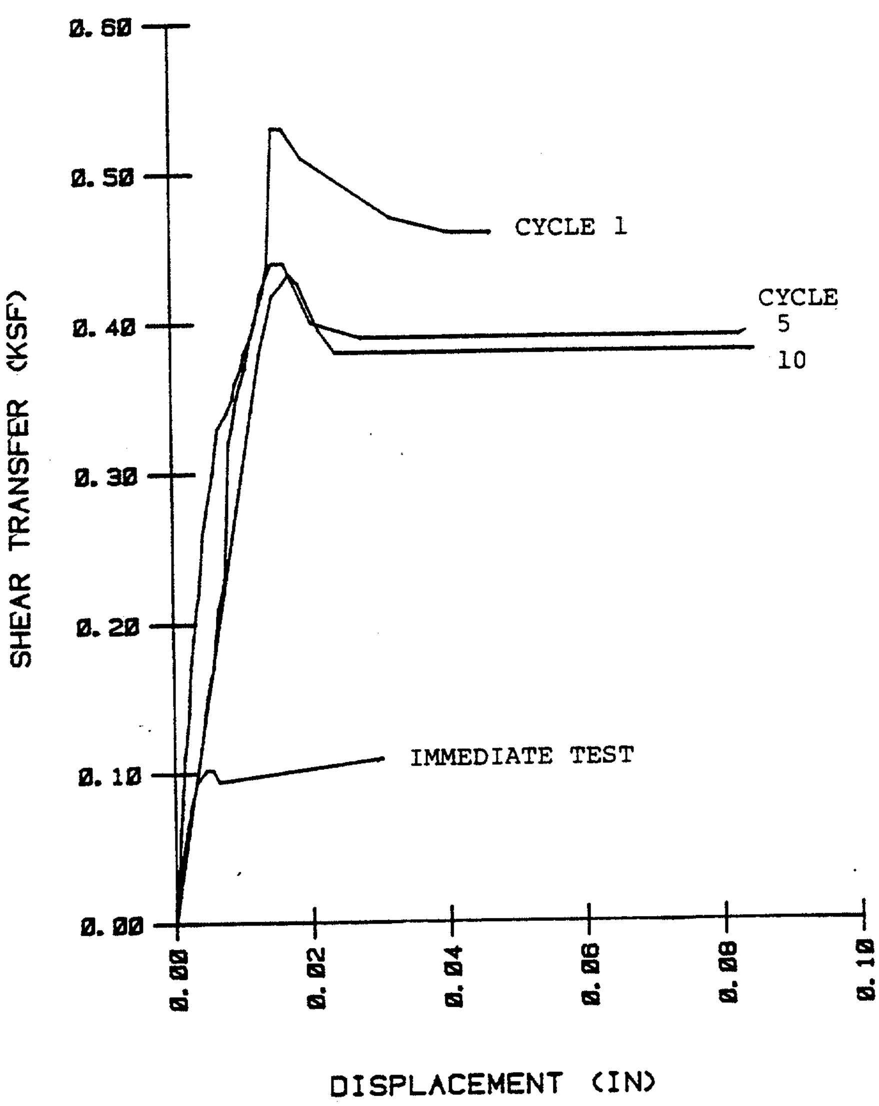
EFFECTS OF LOAD RATE ON THE SOIL BEHAVIOR RECORDED IN EXPERIMENT 1 AT 178 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 2 AT 178 FEET

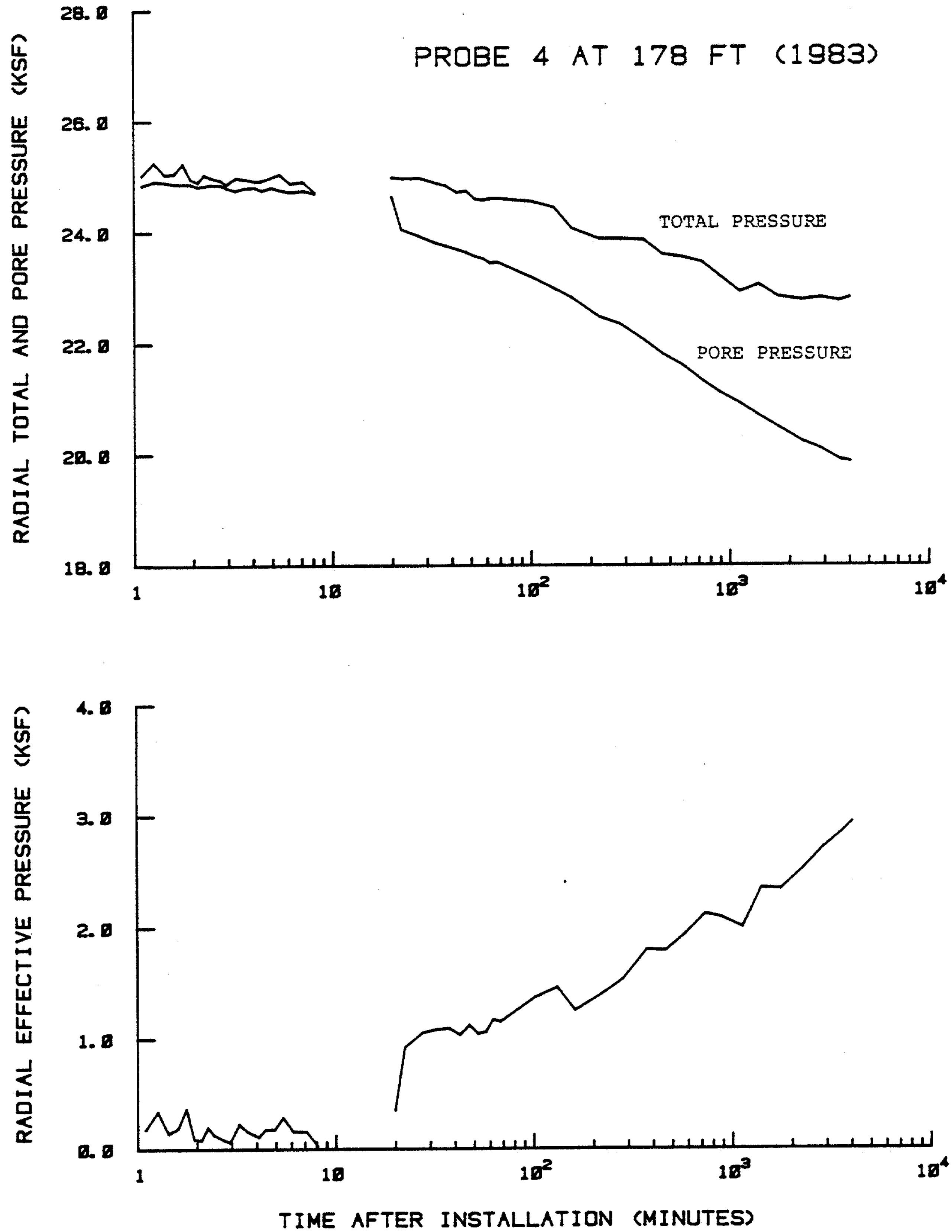
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 178 FT - 8 HR TEST

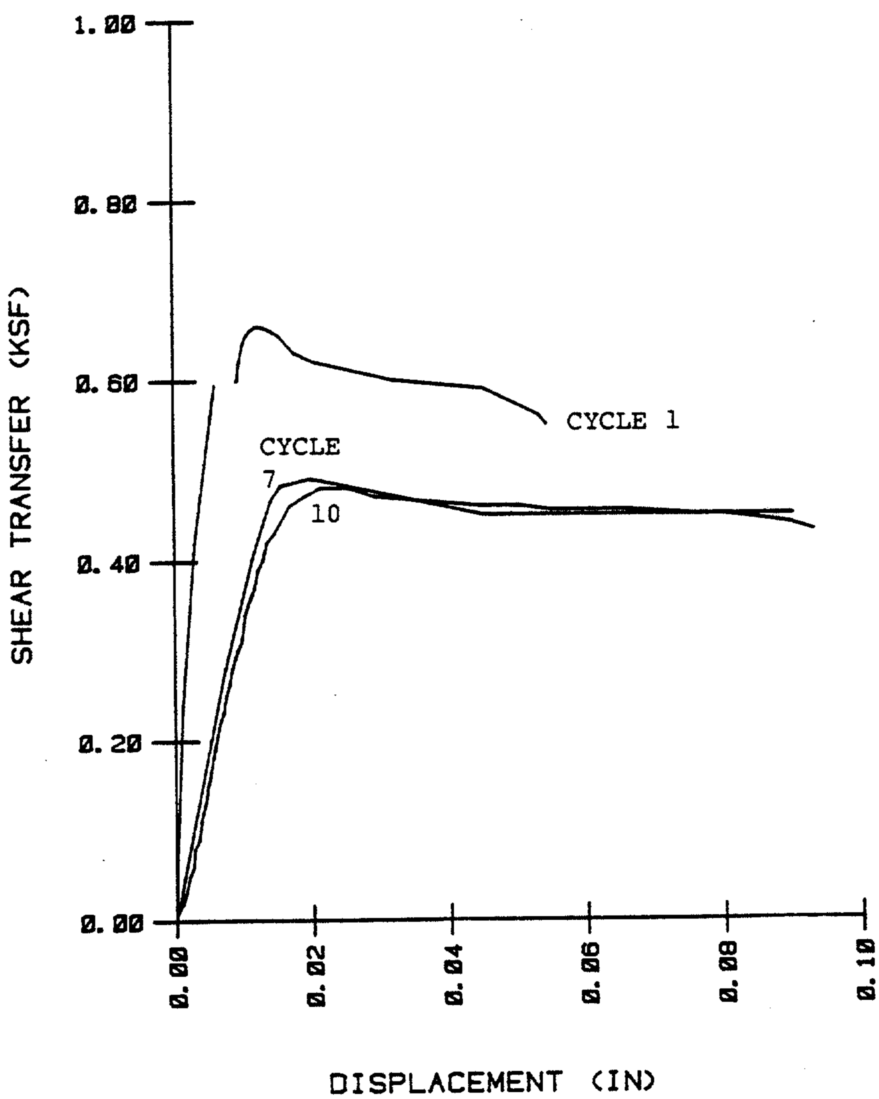
SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 2 AT 178 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



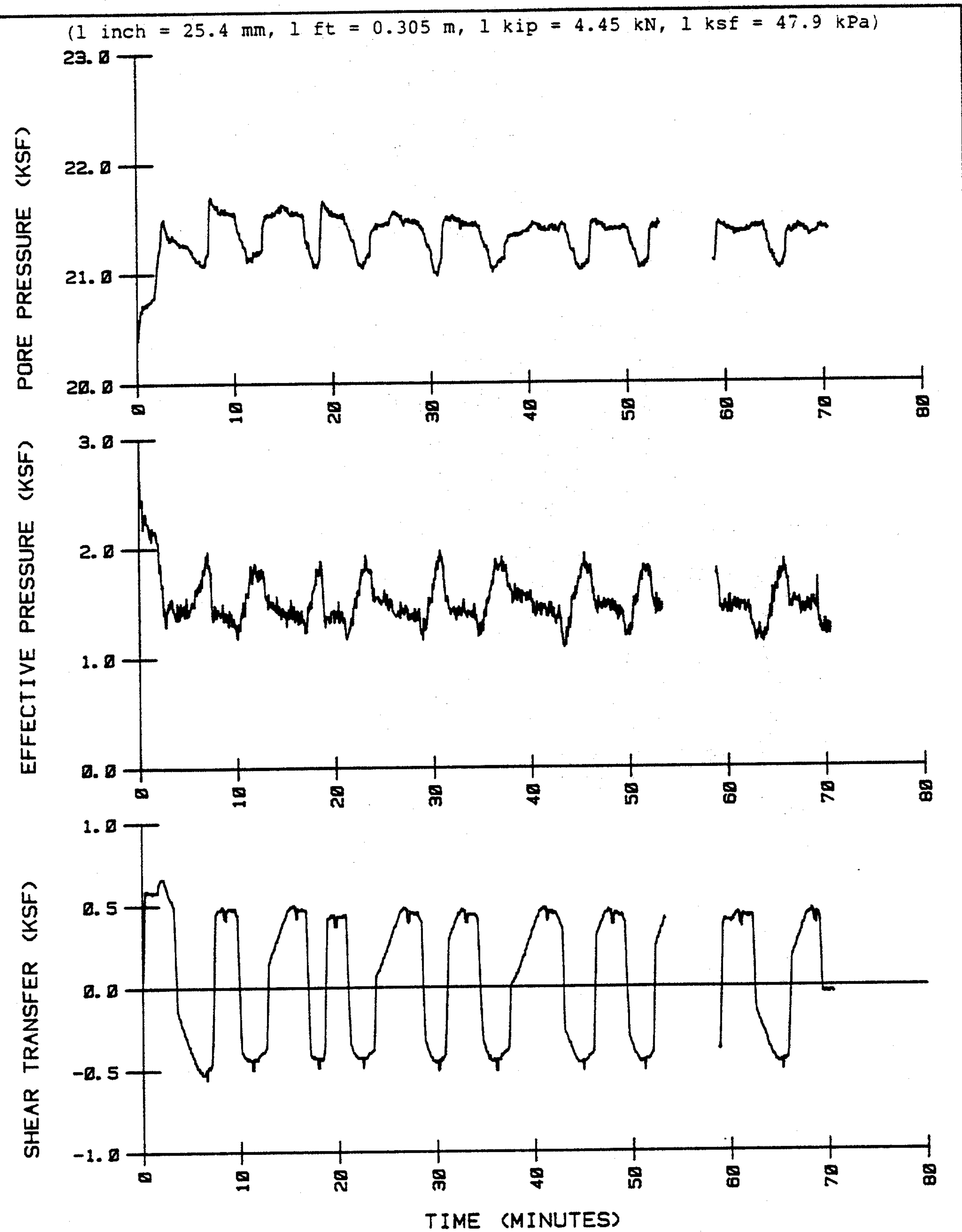
SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 3 AT 178 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 178 FT - 72 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 3 AT 178 FEET

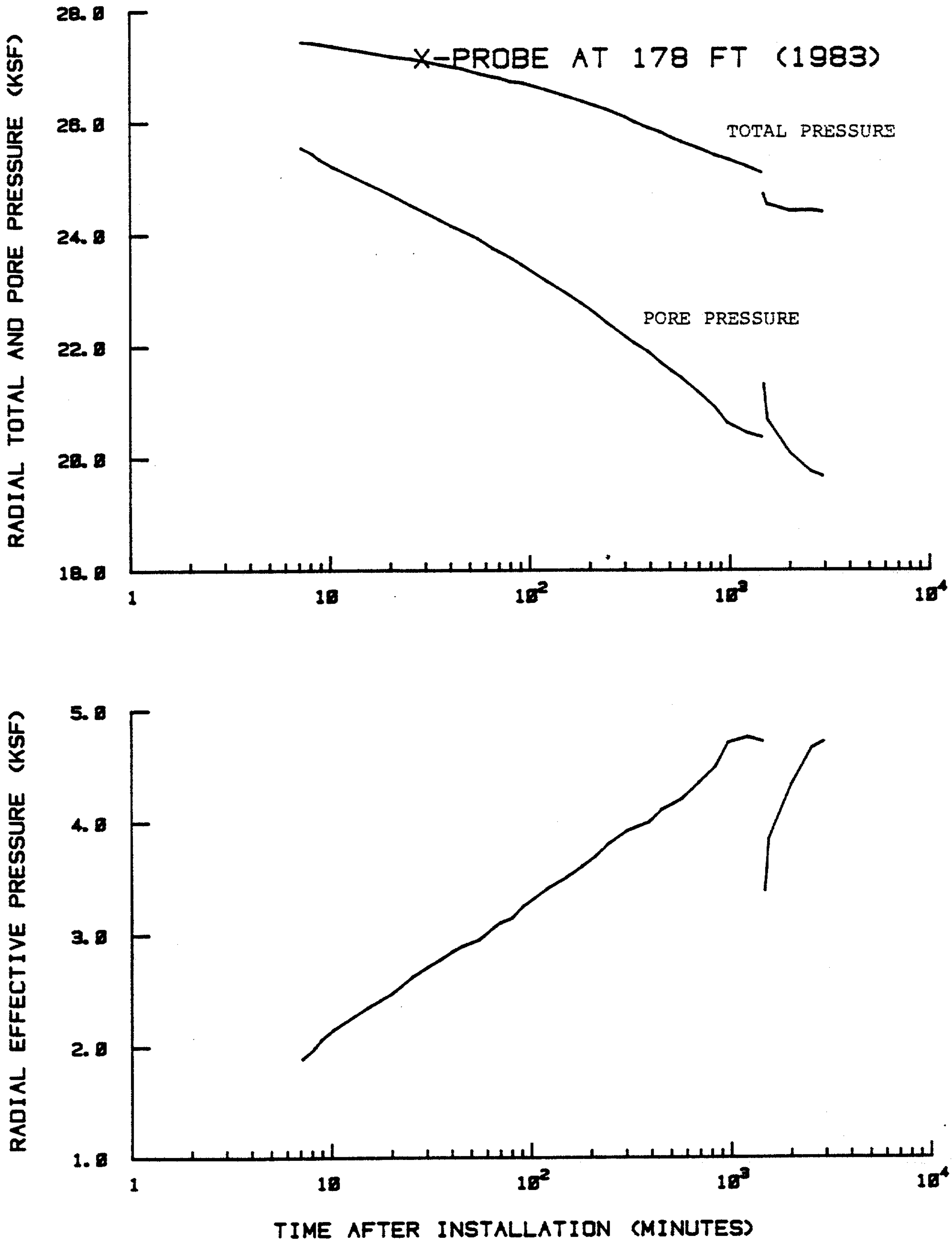


PROBE 4 AT 178 FT - 72 HR TEST

SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 3 AT 178 FEET

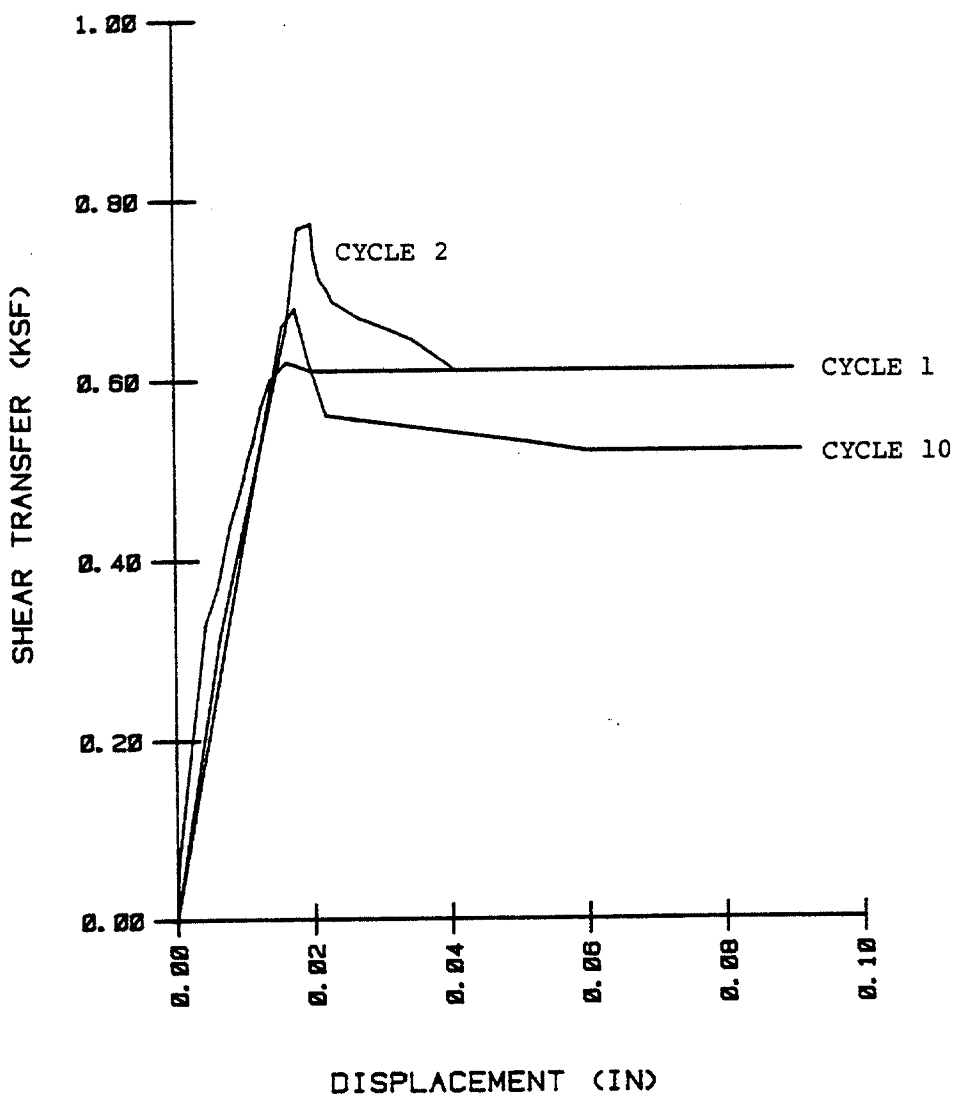
PLATE 5.58

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 4 AT 178 FEET

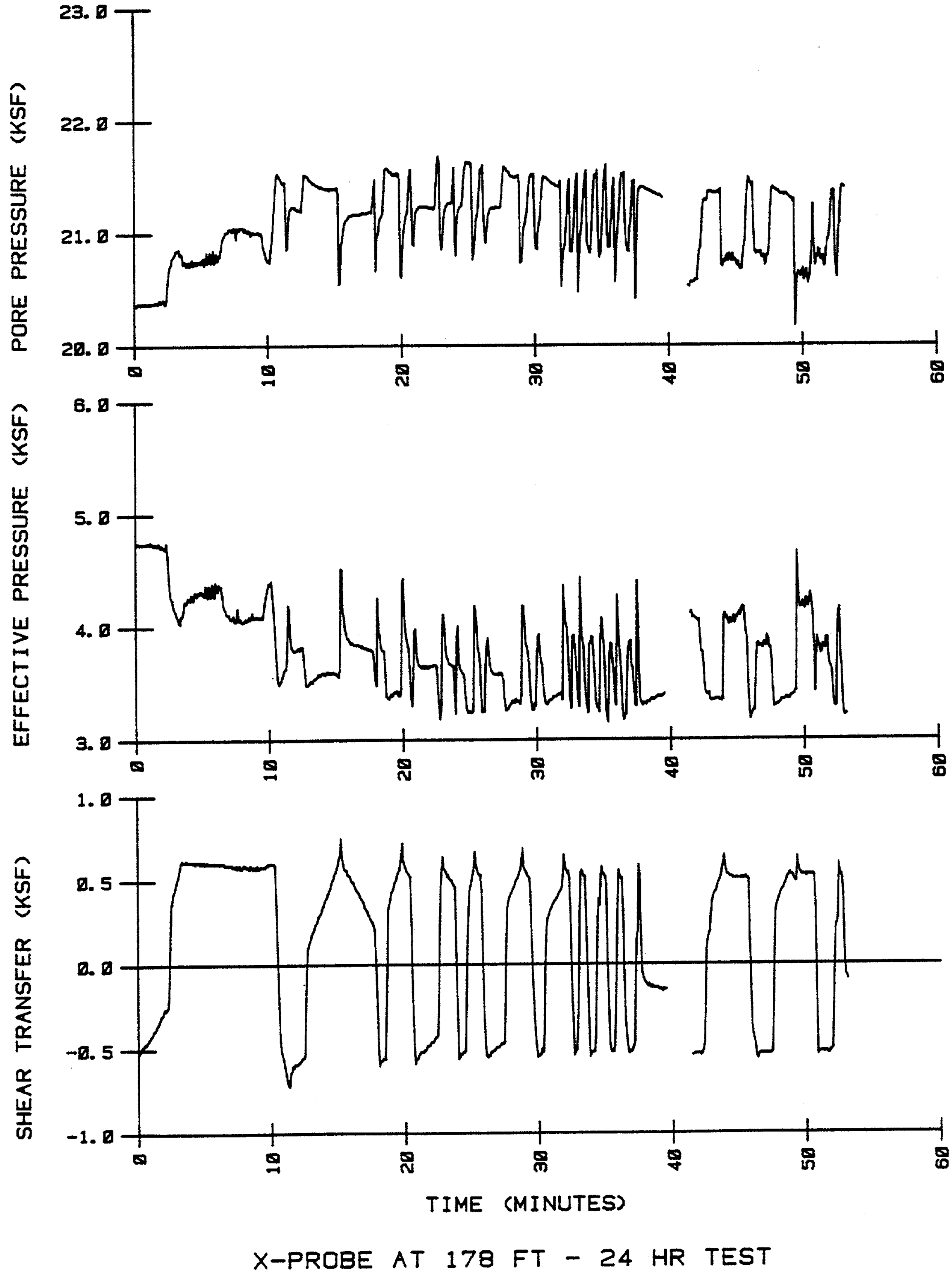
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



X-PROBE AT 178 FT - 24 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 4 AT 178 FEET

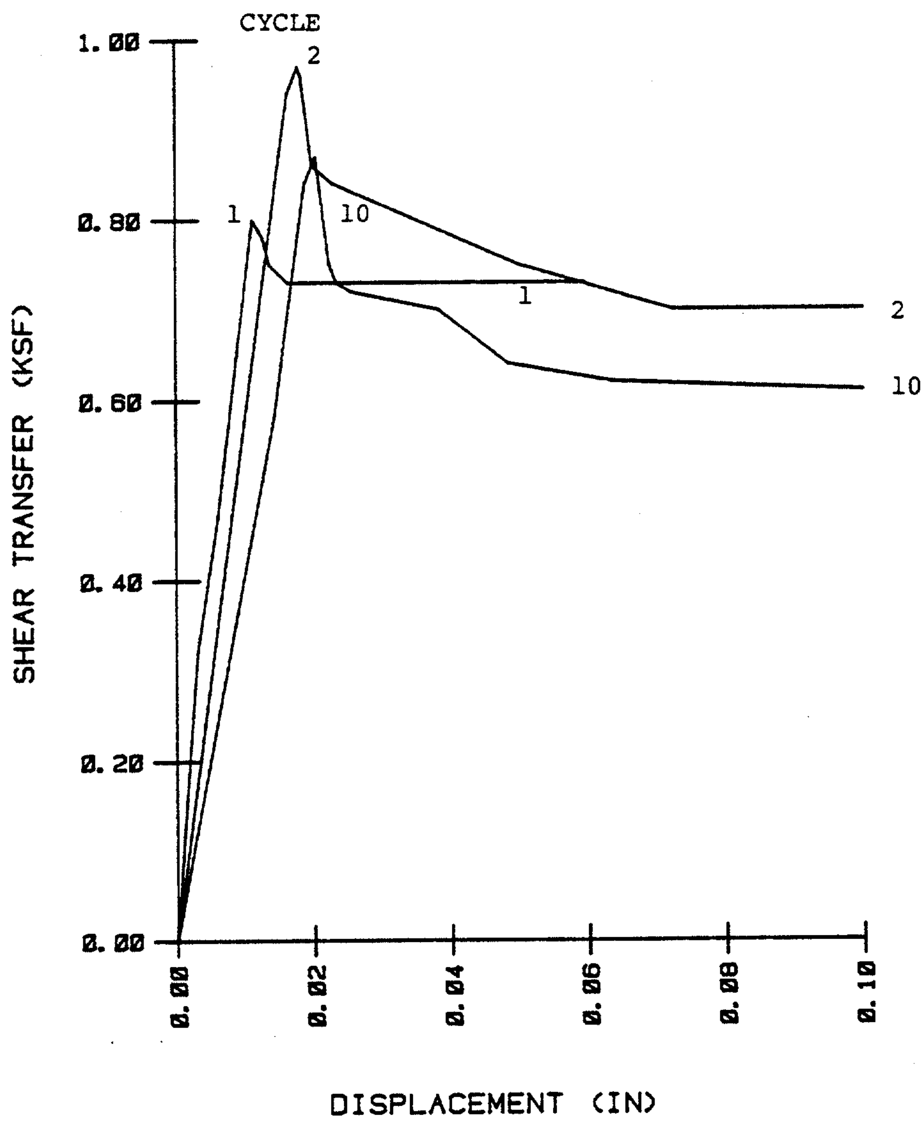
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 4 AT 178 FEET

PLATE 5.61

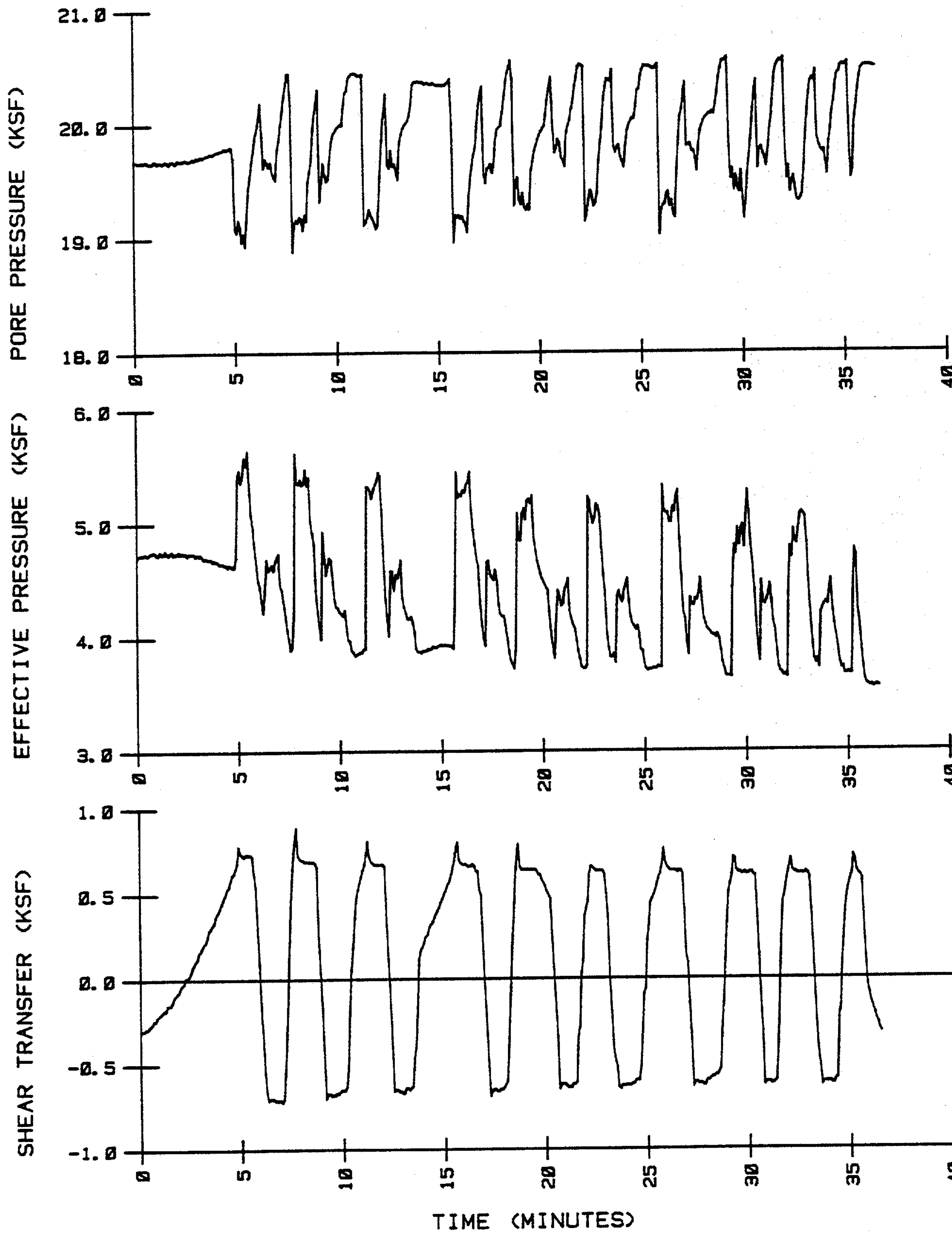
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



X-PROBE AT 178 FT - 49 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 4 AT 178 FEET

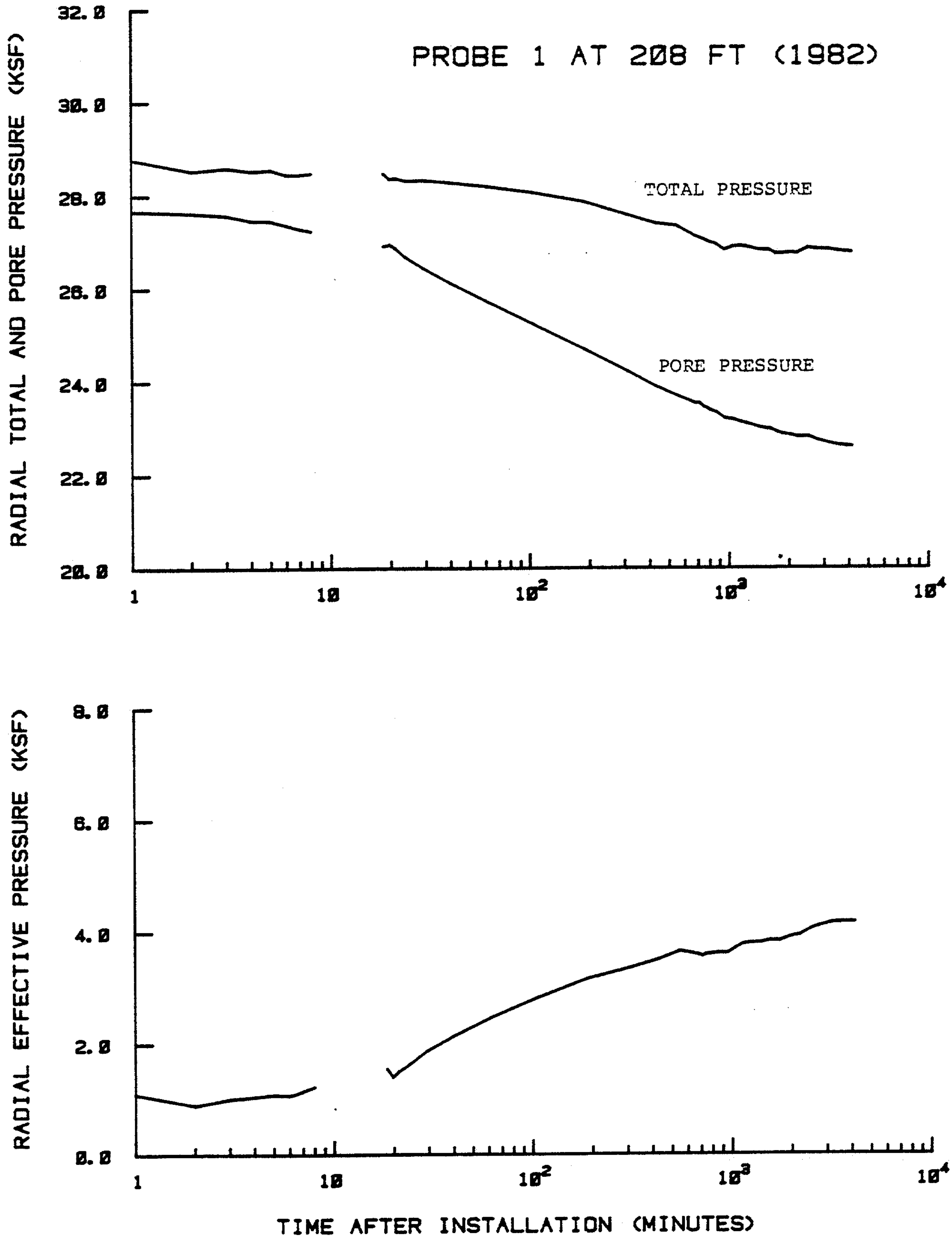
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



X-PROBE AT 178 FT - 49 HR TEST

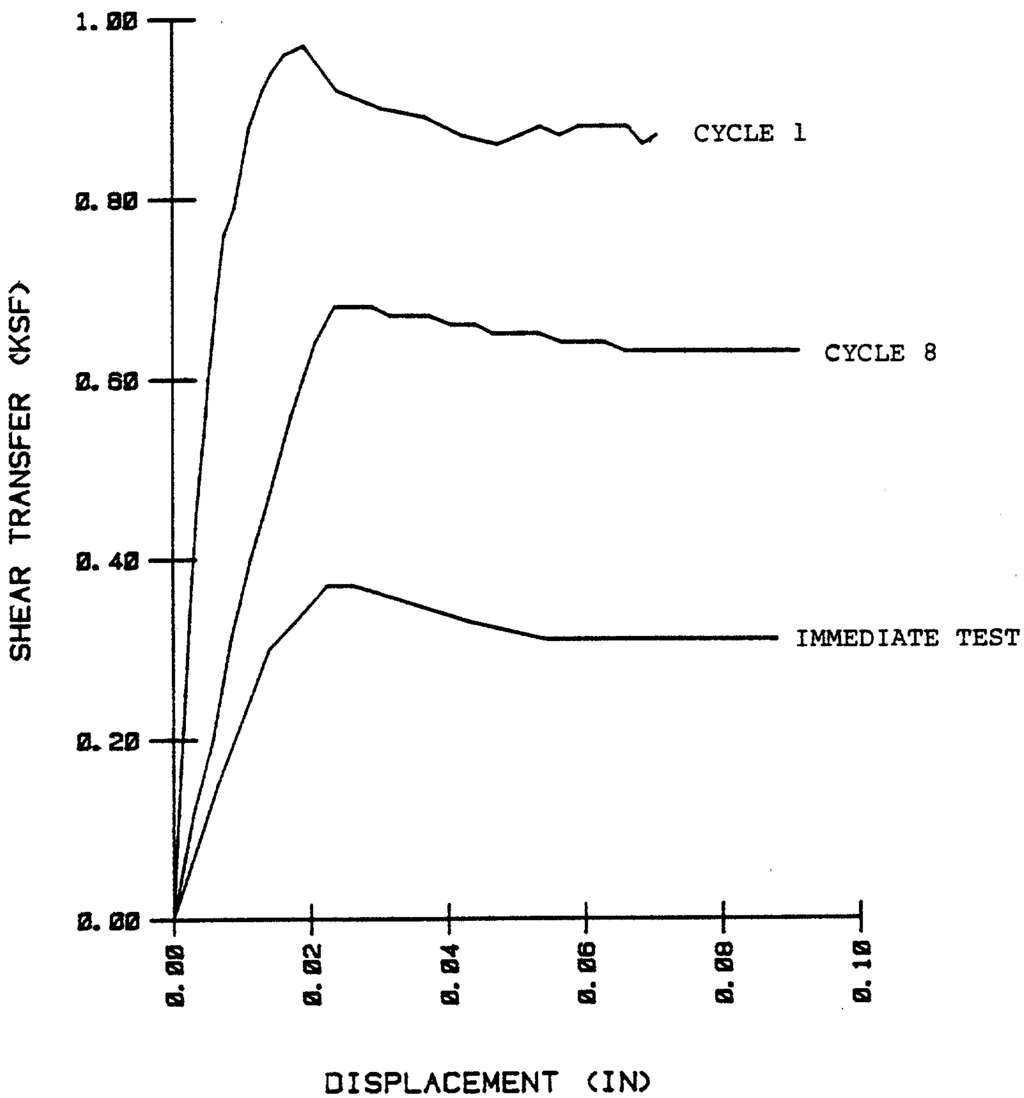
SOIL BEHAVIOR RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 4 AT 178 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 1 AT 208 FEET

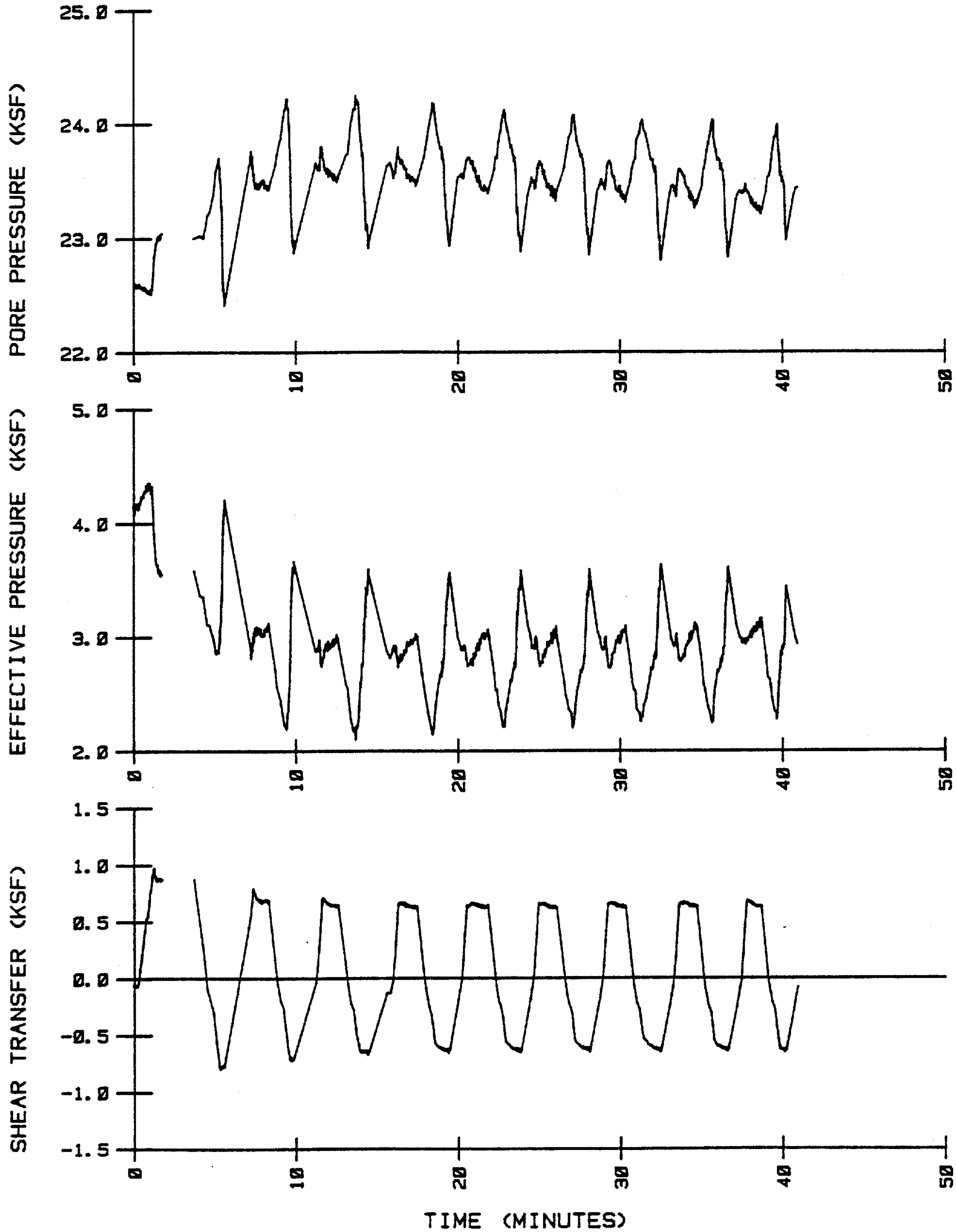
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 208 FT - 71 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 1 AT 208 FEET

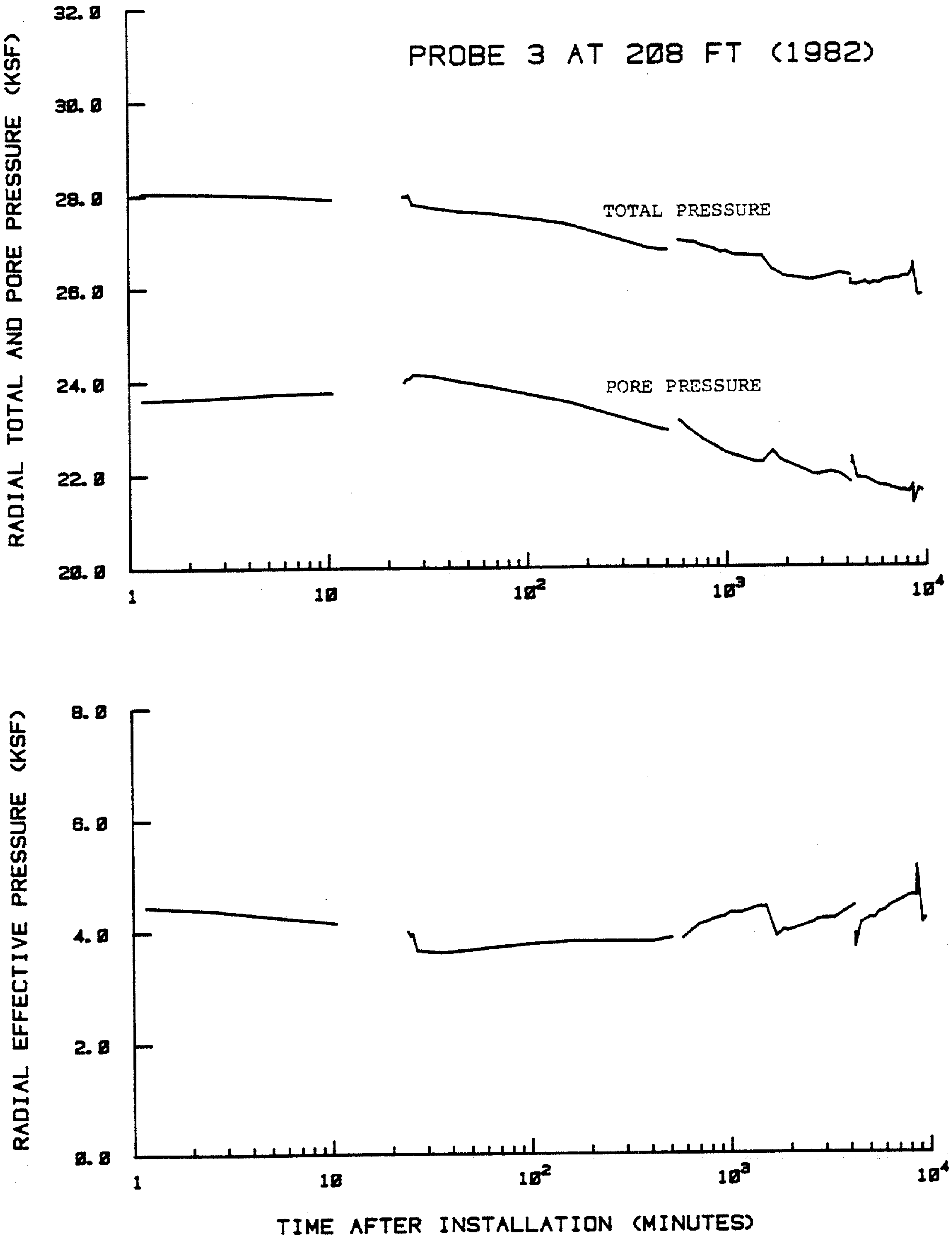
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 208 FT - 71 HR TEST

SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 1 AT 208 FEET

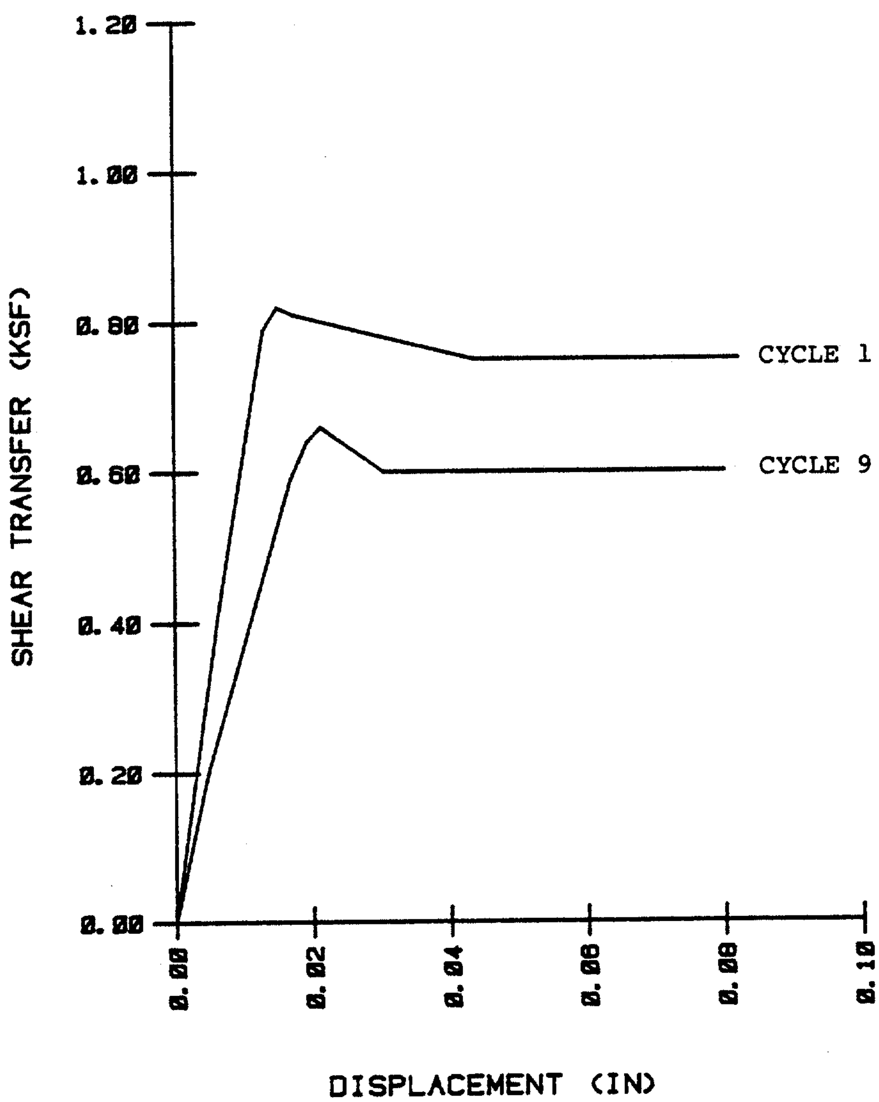
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 2 AT 208 FEET

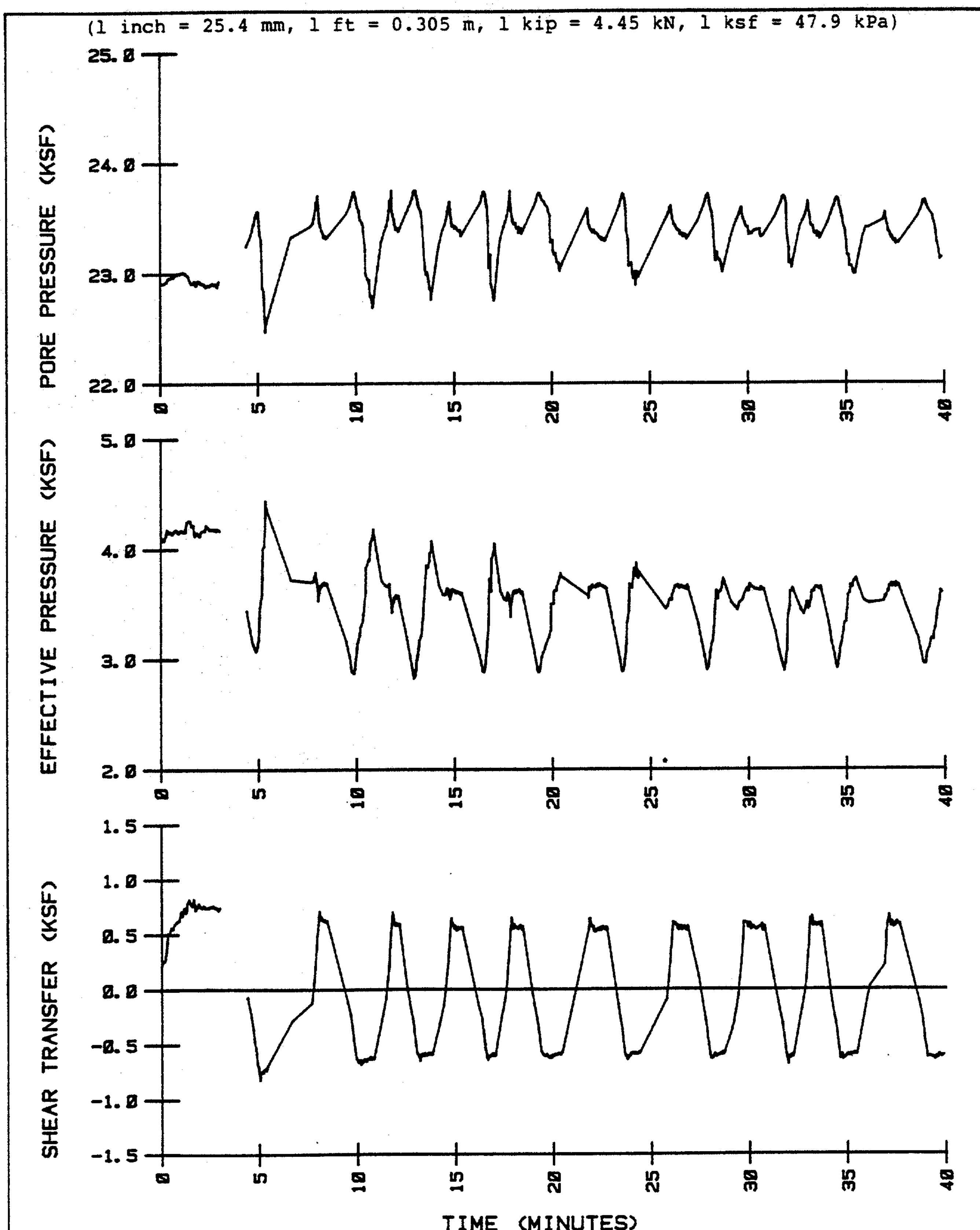
PLATE 5.67

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 208 FT - 8 HR TEST

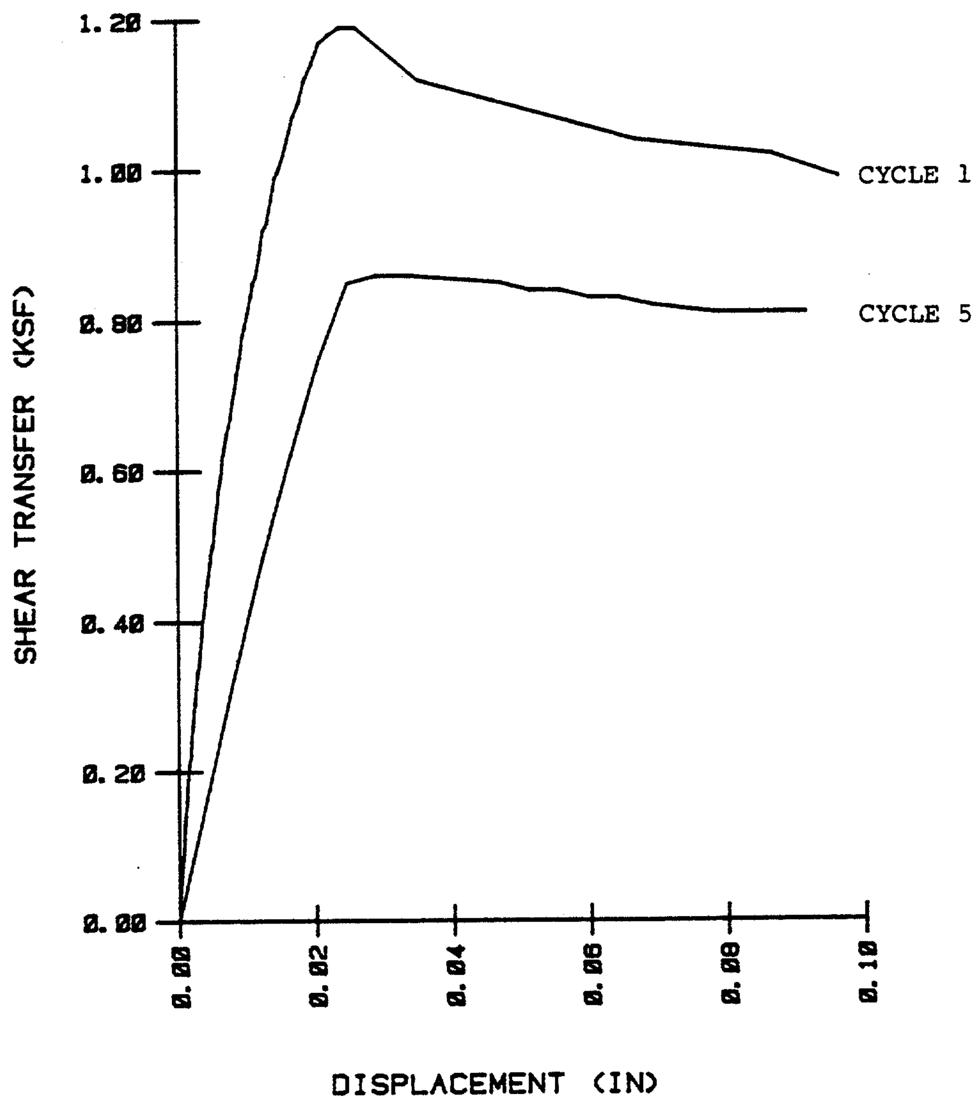
SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 2 AT 208 FEET



SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 2 AT 208 FEET

PLATE 5.69

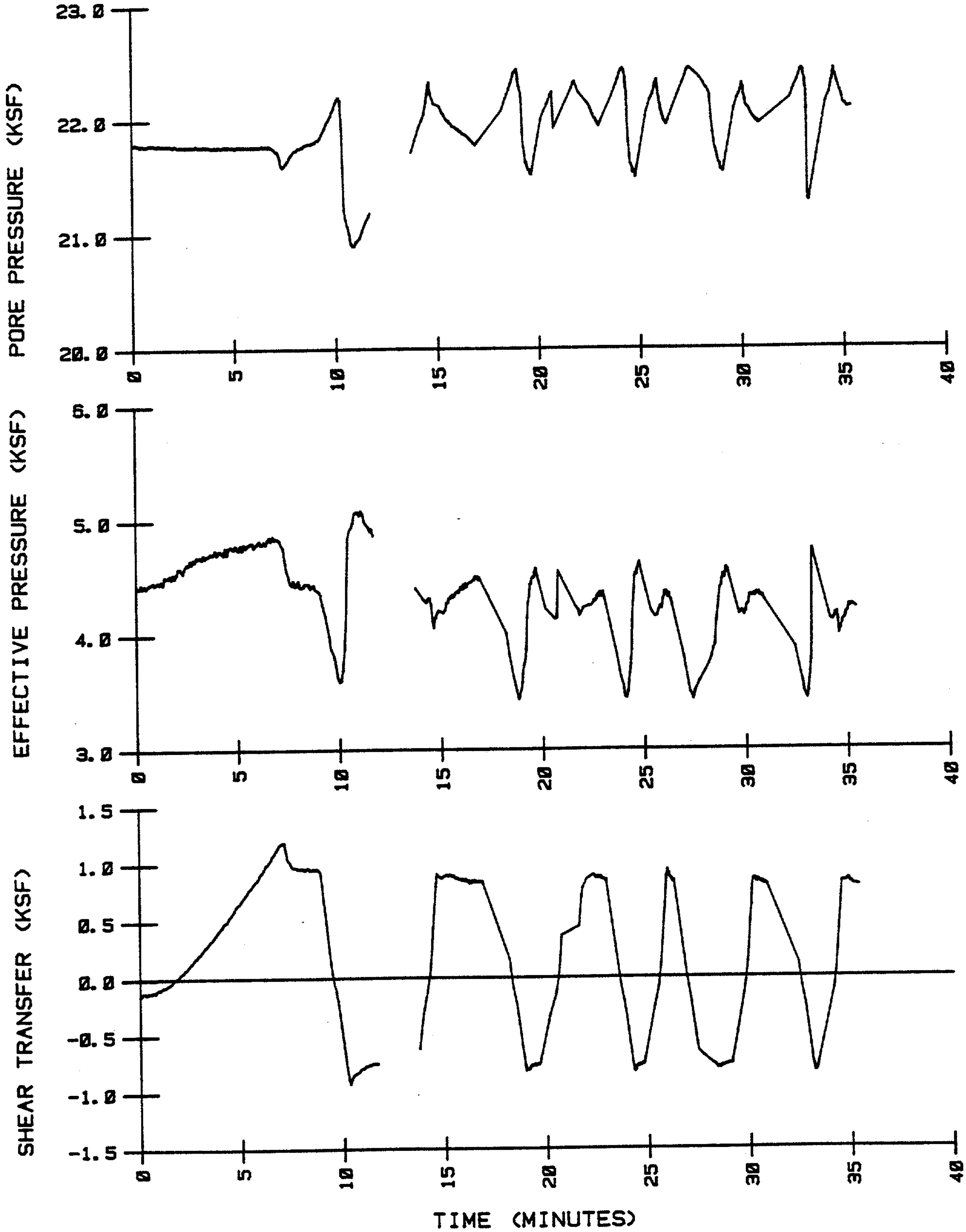
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 208 FT - 70 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 2 AT 208 FEET

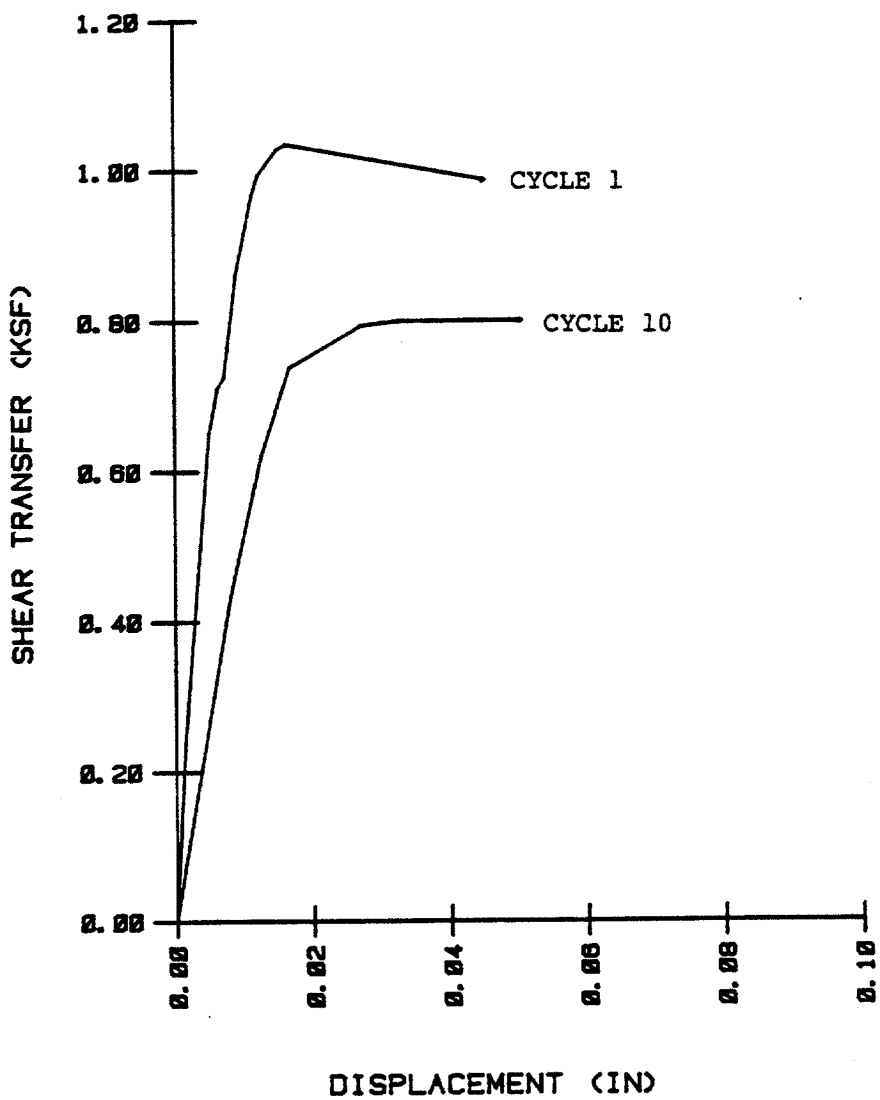
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 208 FT - 70 HR TEST

SOIL BEHAVIOR RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 2 AT 208 FEET

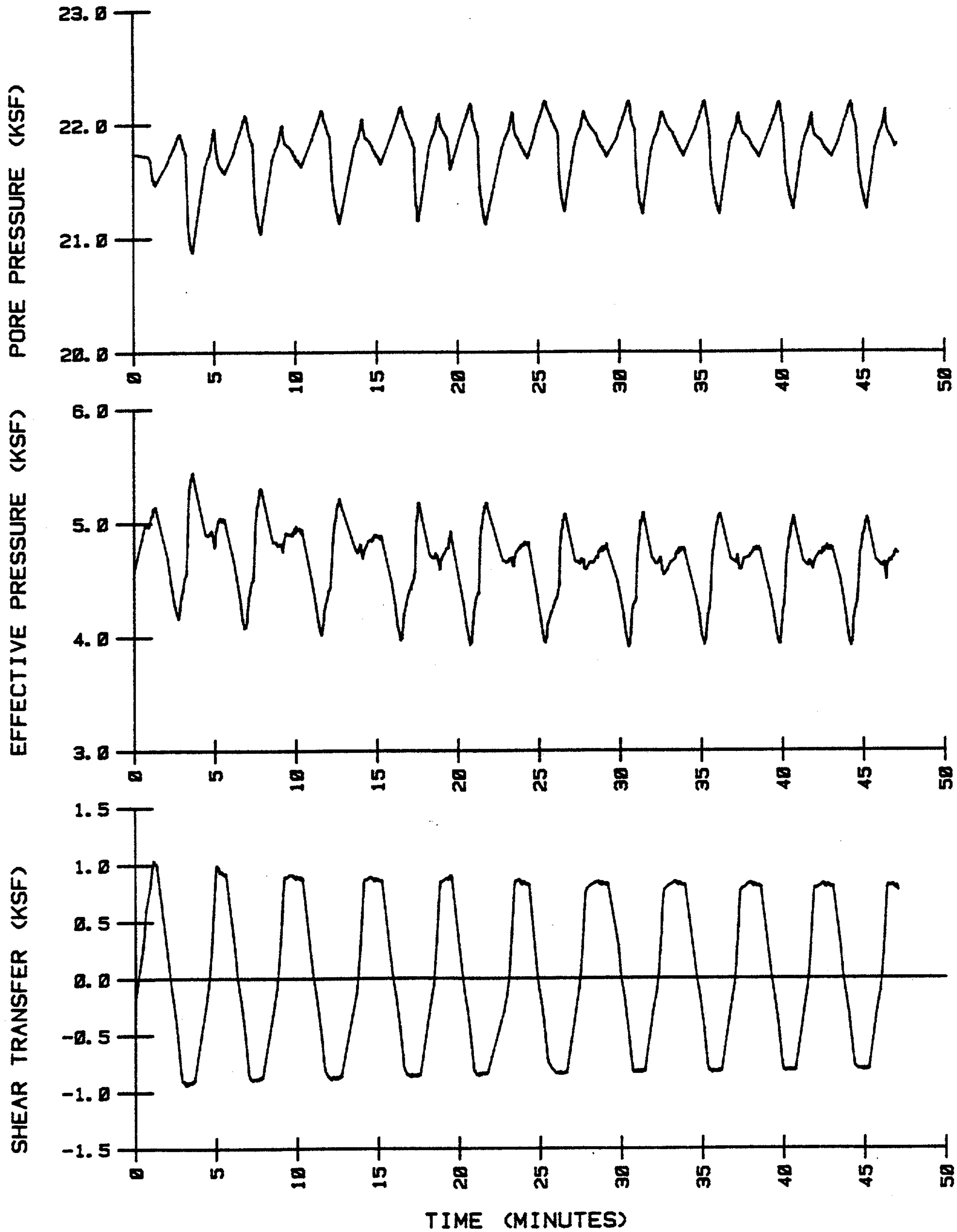
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 208 FT - 143 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE THIRD CYCLIC TEST IN EXPERIMENT 2 AT 208 FEET

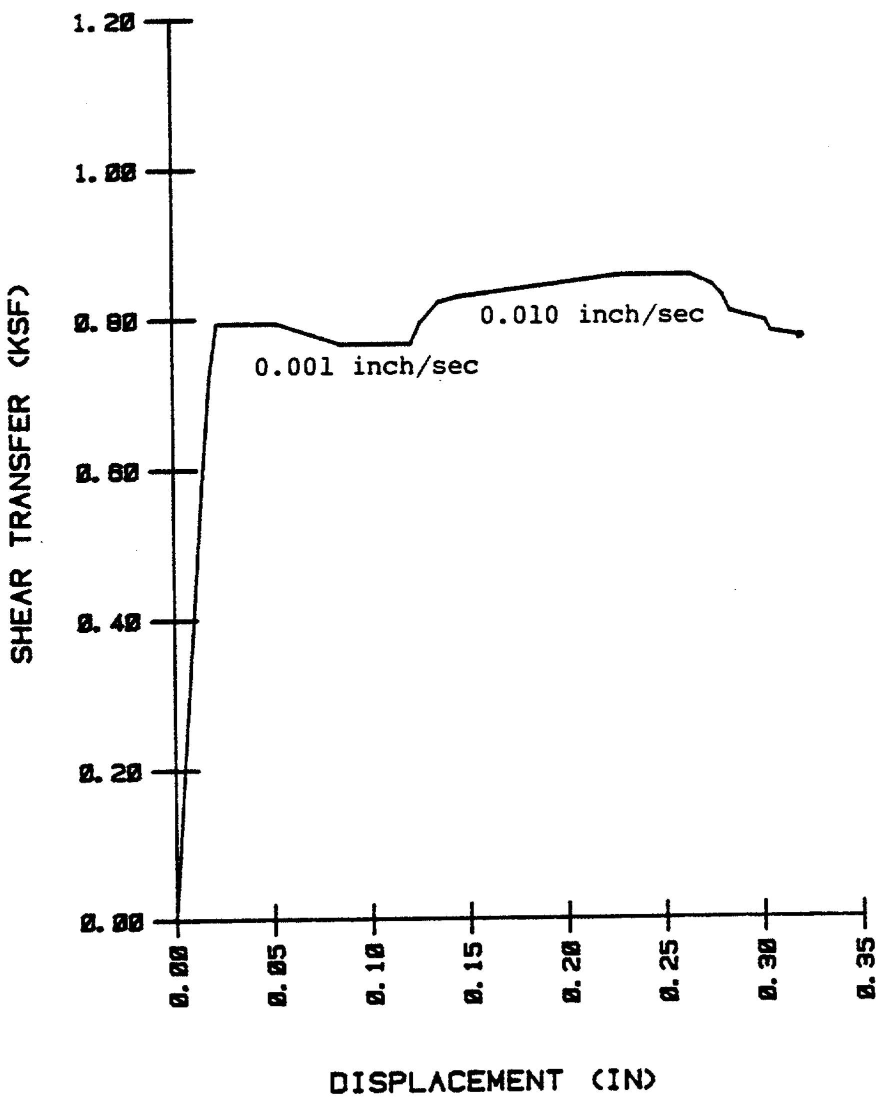
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 208 FT - 143 HR TEST

SOIL BEHAVIOR RECORDED DURING THE THIRD CYCLIC TEST IN EXPERIMENT 2 AT 208 FEET

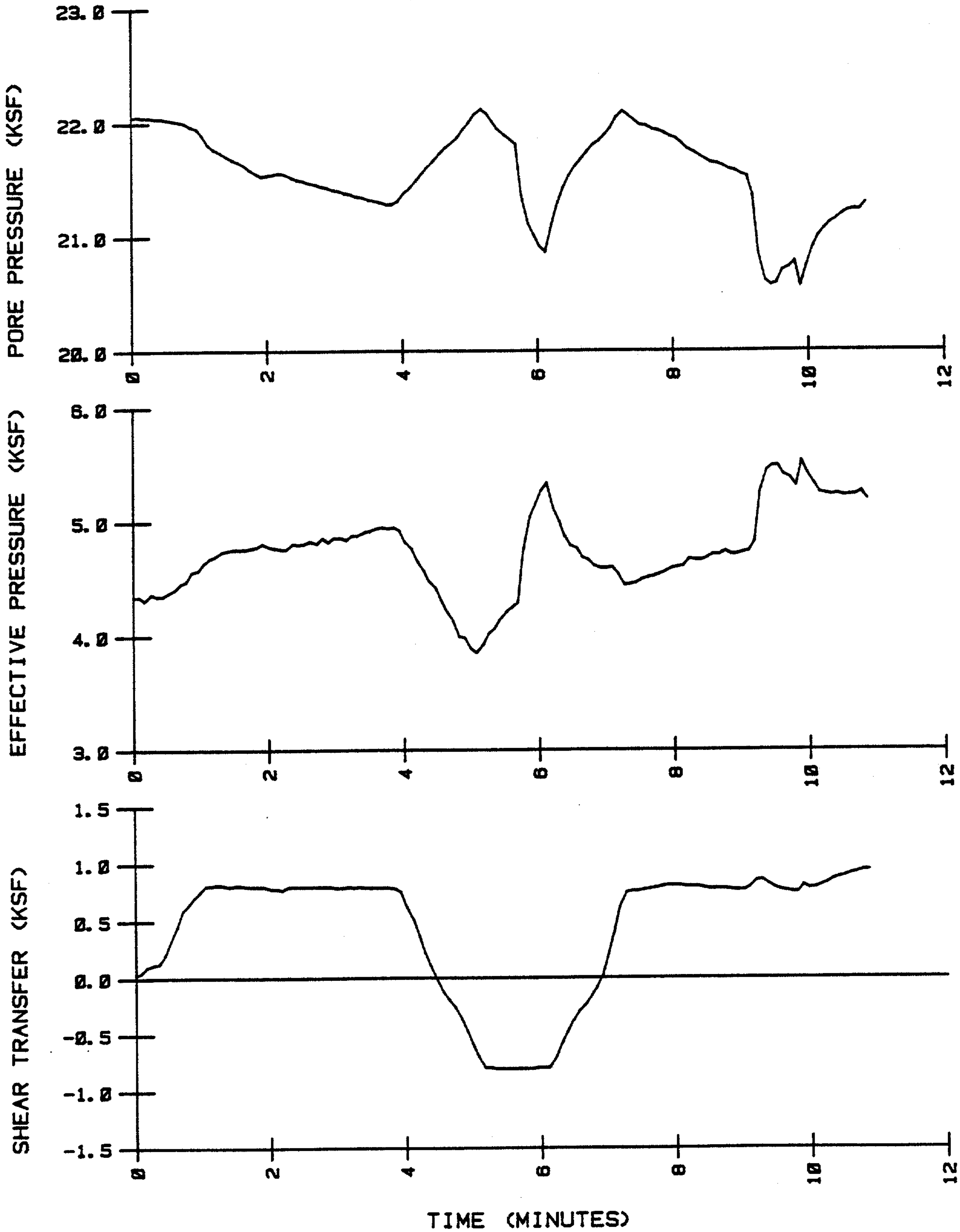
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 208 FT - PULL OUT TEST

EFFECTS OF LOAD RATE ON THE SHEAR TRANSFER RECORDED
DURING EXPERIMENT 2 AT 208 FEET

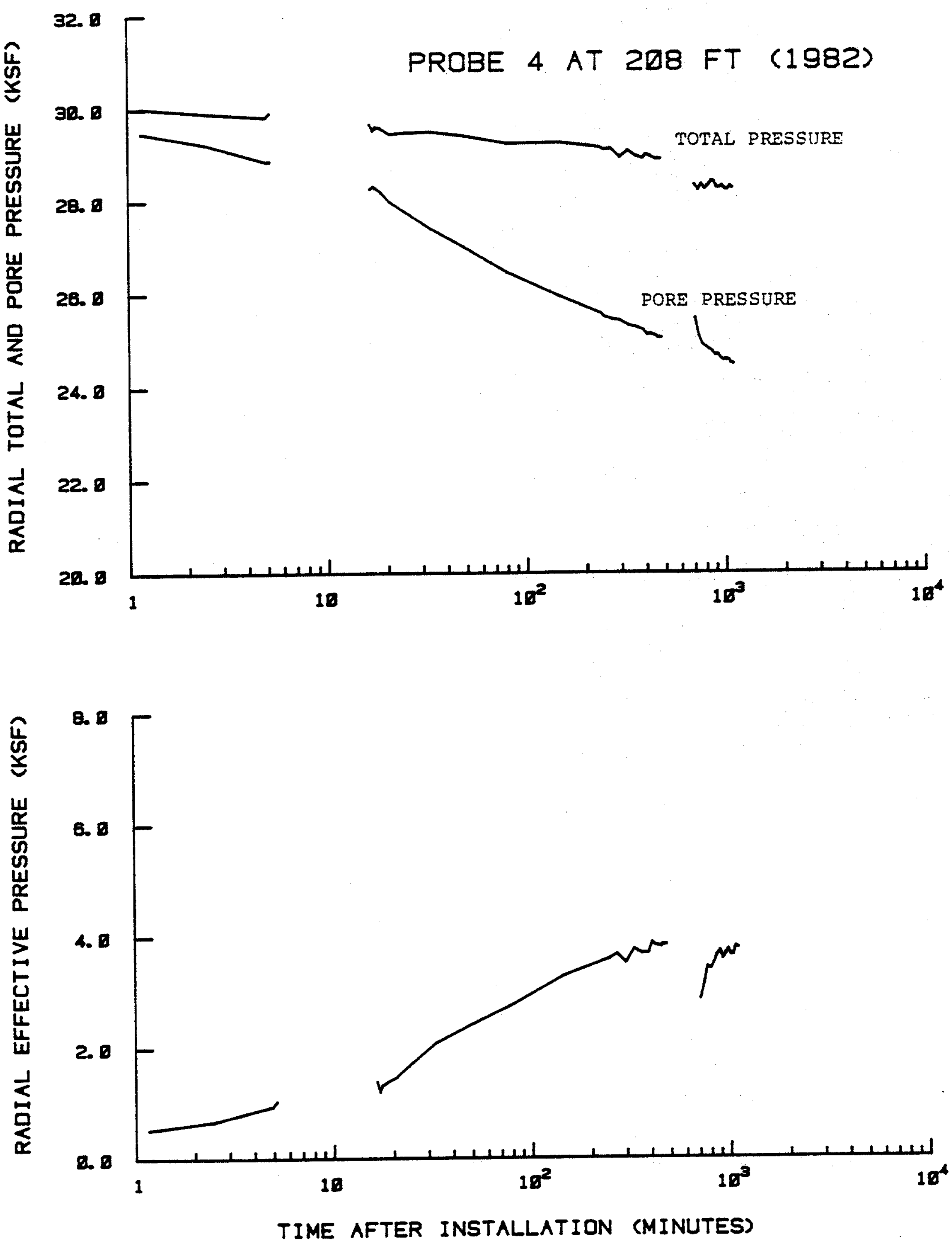
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 208 FT - PULL OUT TEST

EFFECTS OF LOAD RATE ON THE SOIL BEHAVIOR RECORDED
DURING EXPERIMENT 2 AT 208 FEET

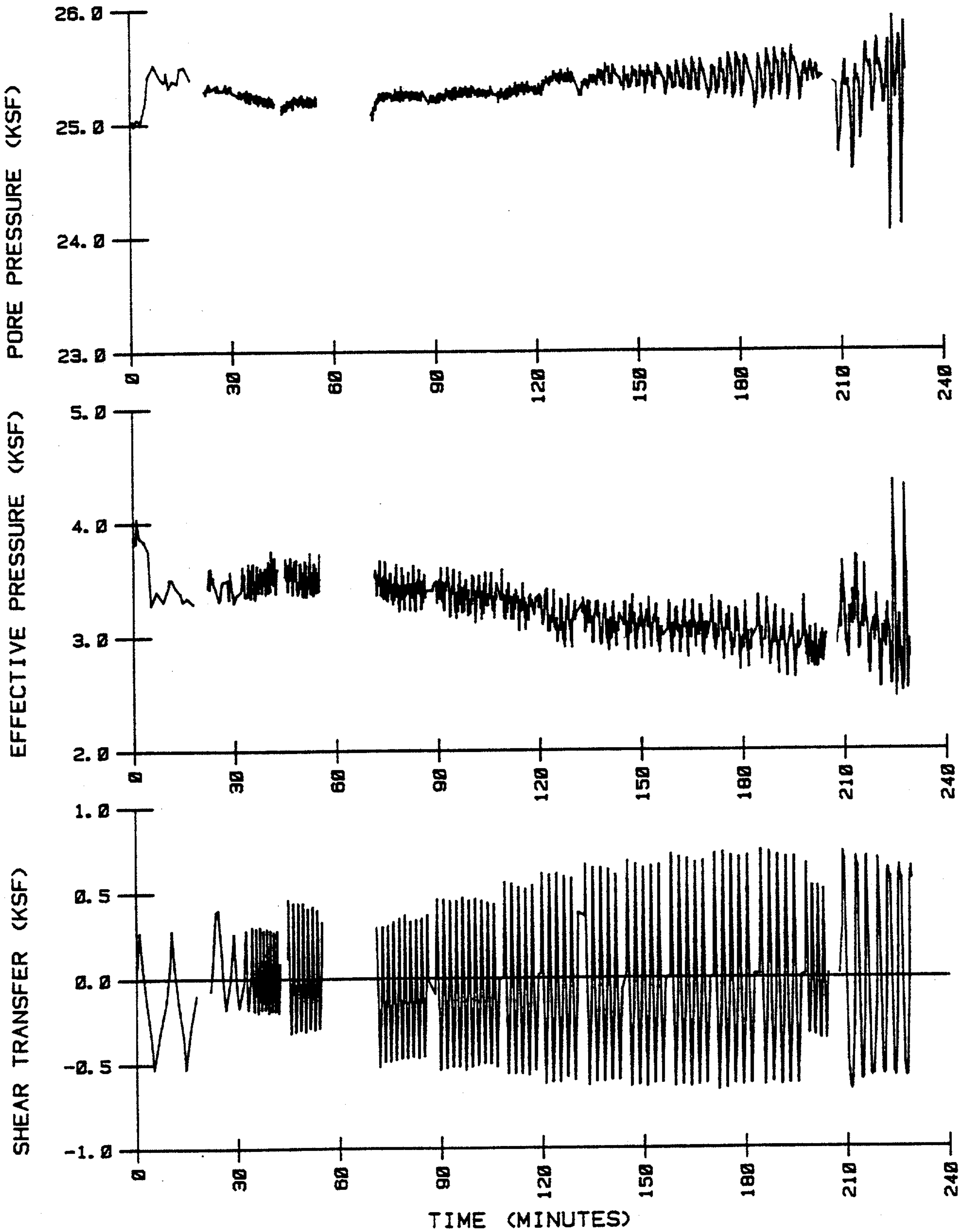
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 3 AT 208 FEET

PLATE 5.76

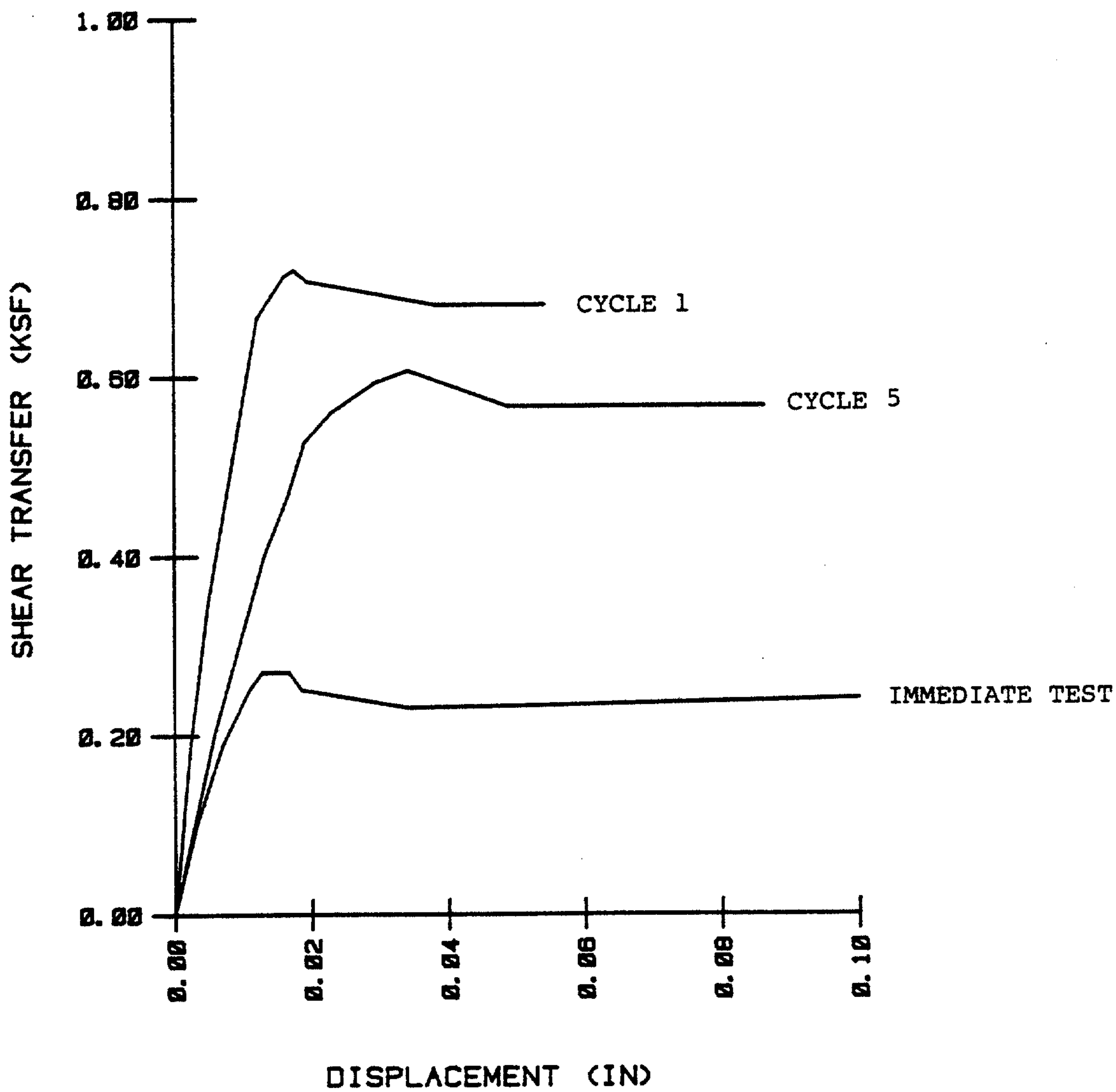
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 208 FT - 8 HR TEST

SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 3 AT 208 FEET

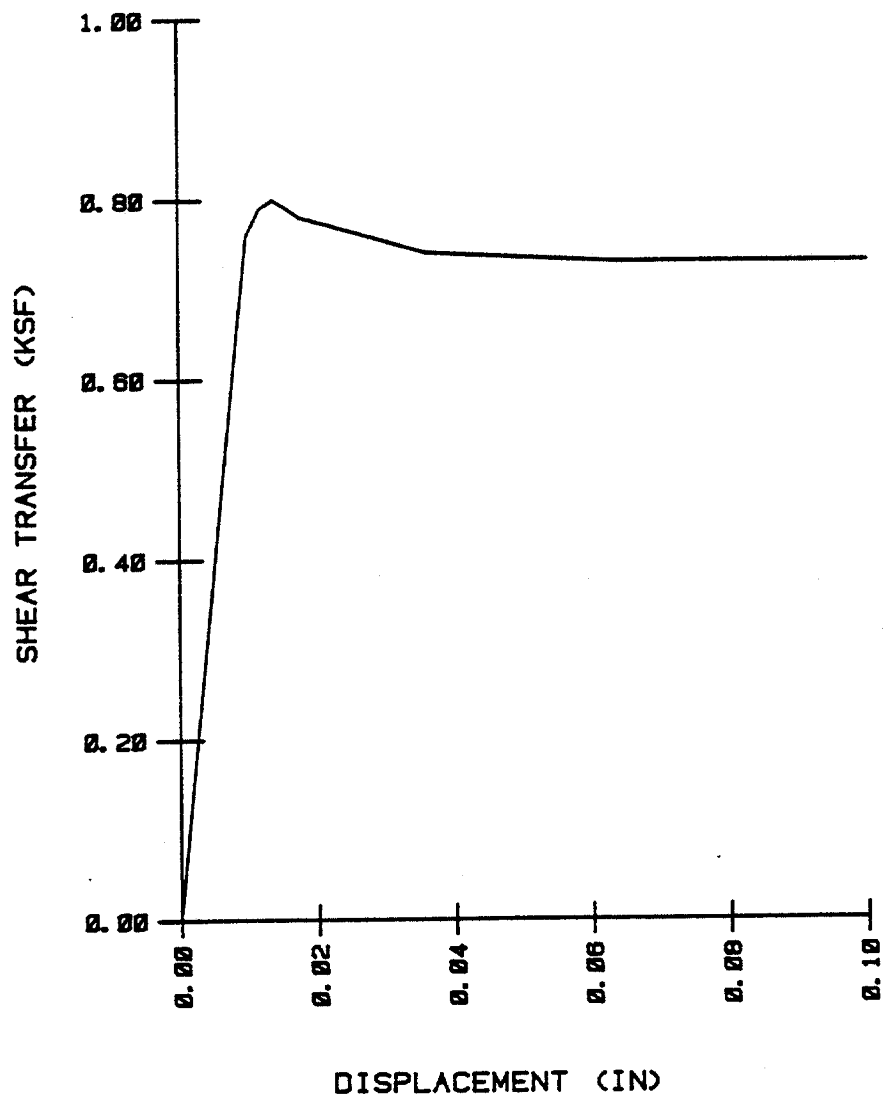
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 208 FT - 8 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 3 AT 208 FEET

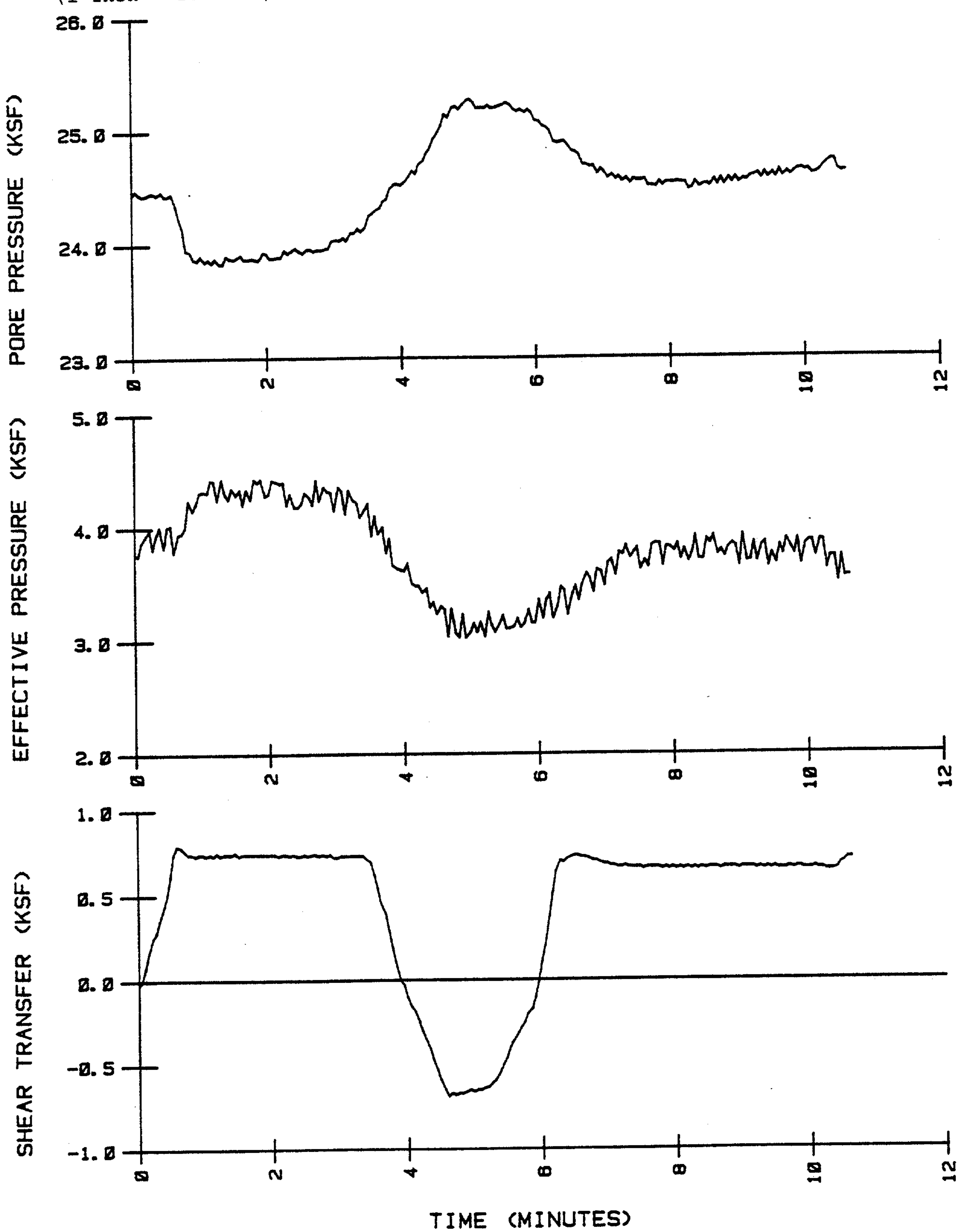
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 208 FT - PULL OUT TEST

SHEAR TRANSFER CURVES RECORDED AFTER ADDITIONAL CONSOLIDATION IN EXPERIMENT 3 AT 208 FEET

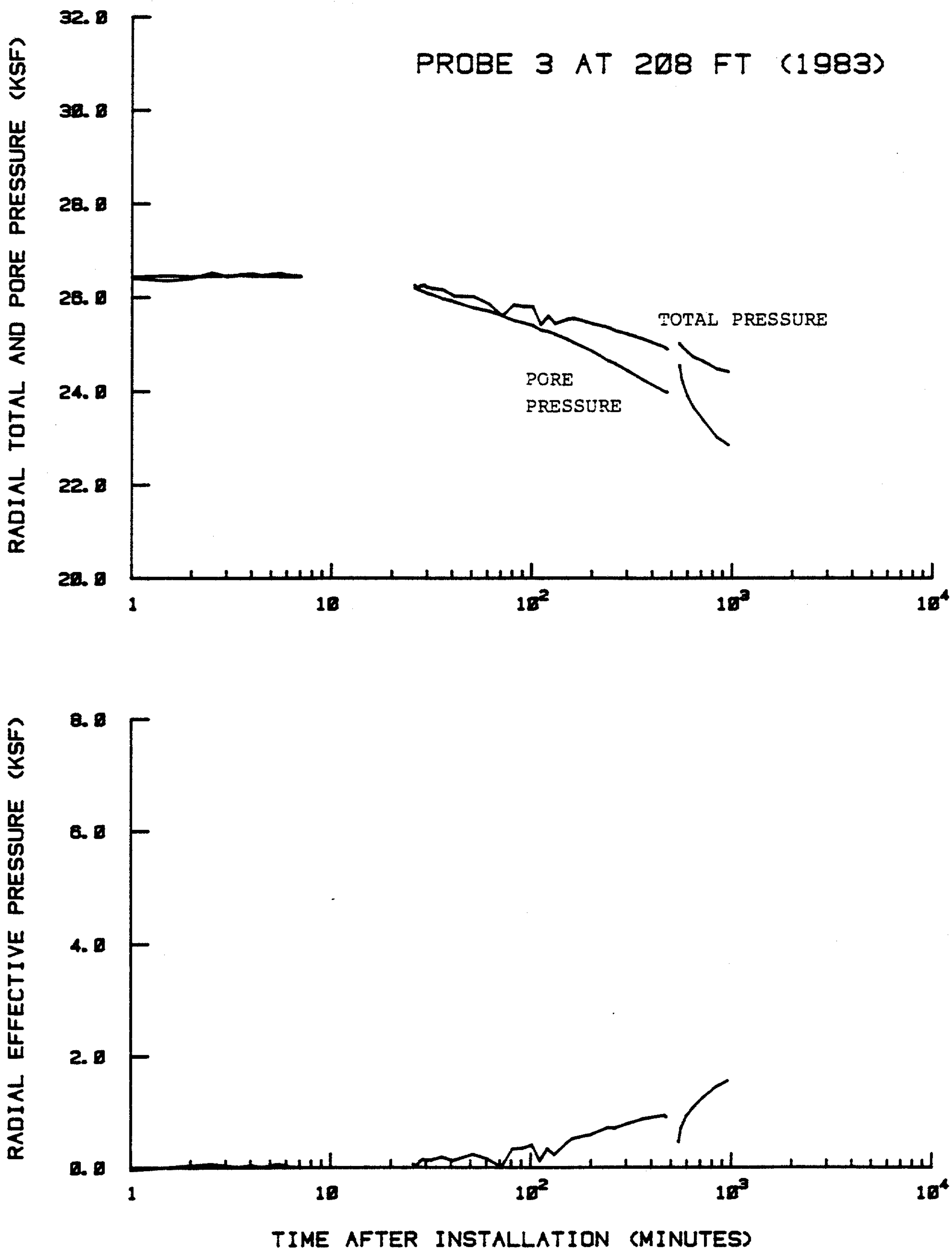
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 4 AT 208 FT - PULL OUT TEST

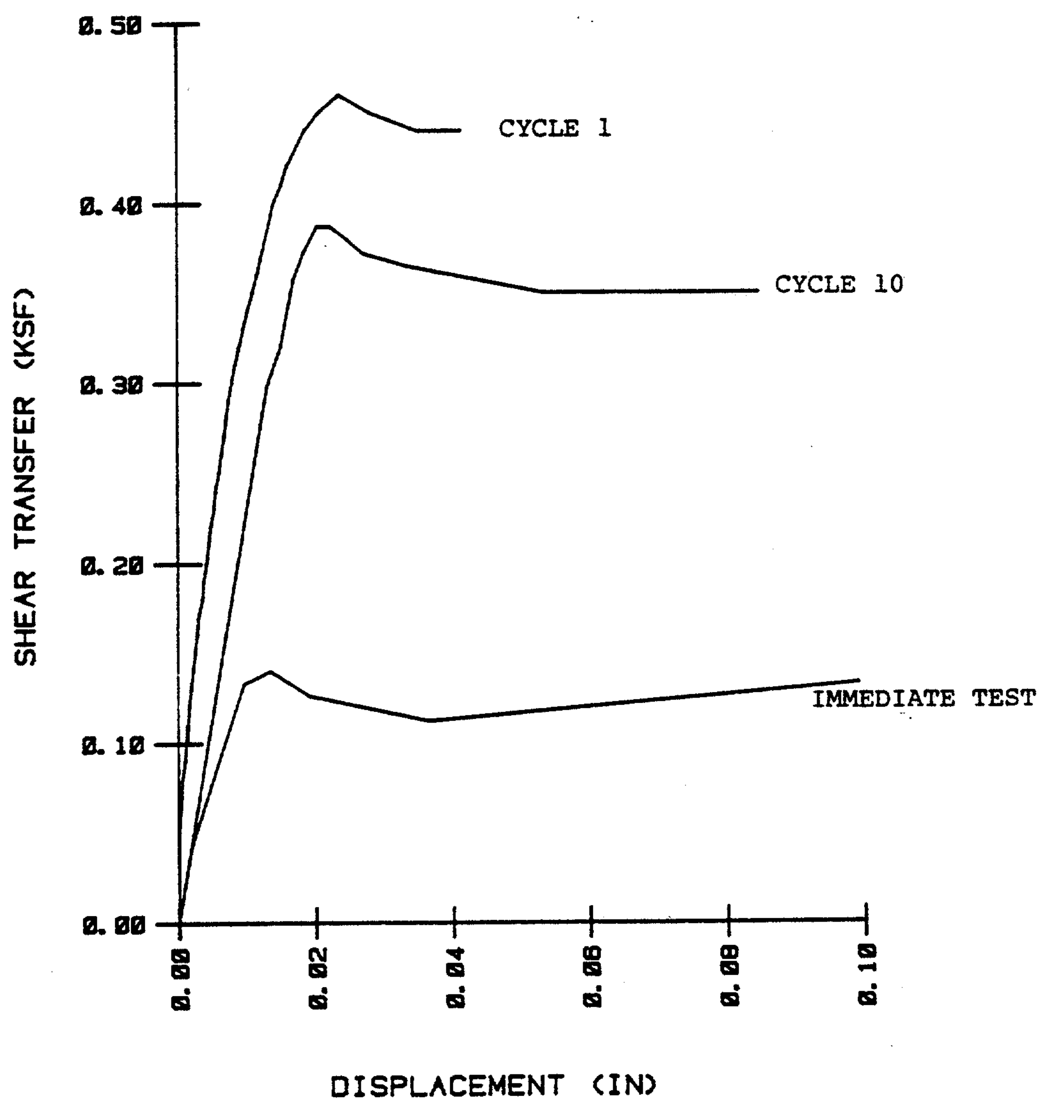
SOIL BEHAVIOR RECORDED DURING THE RETEST AFTER ADDITIONAL CONSOLIDATION IN EXPERIMENT 3 AT 208 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 4 AT 208 FEET

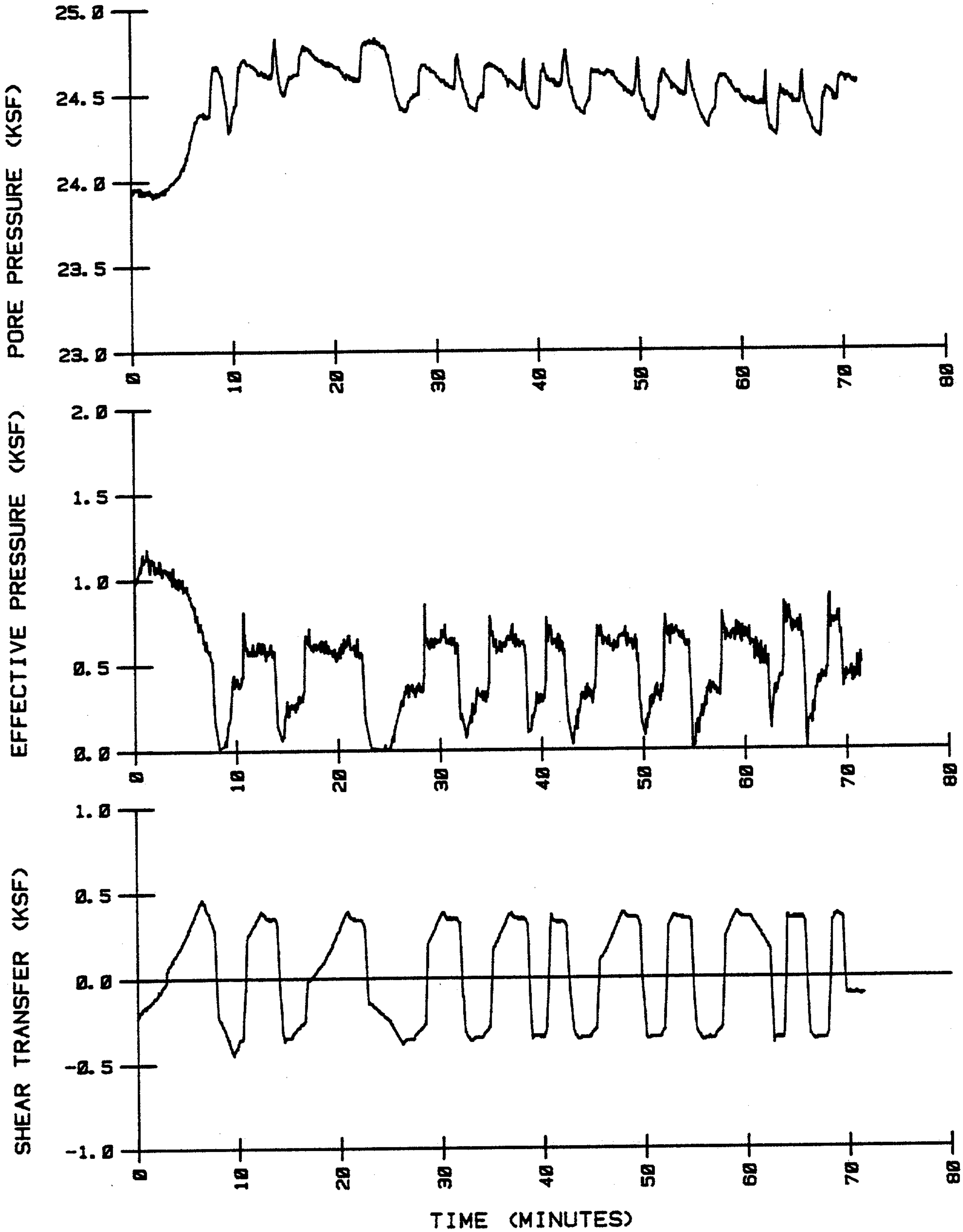
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 208 FT - 8 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 4 AT 208 FEET

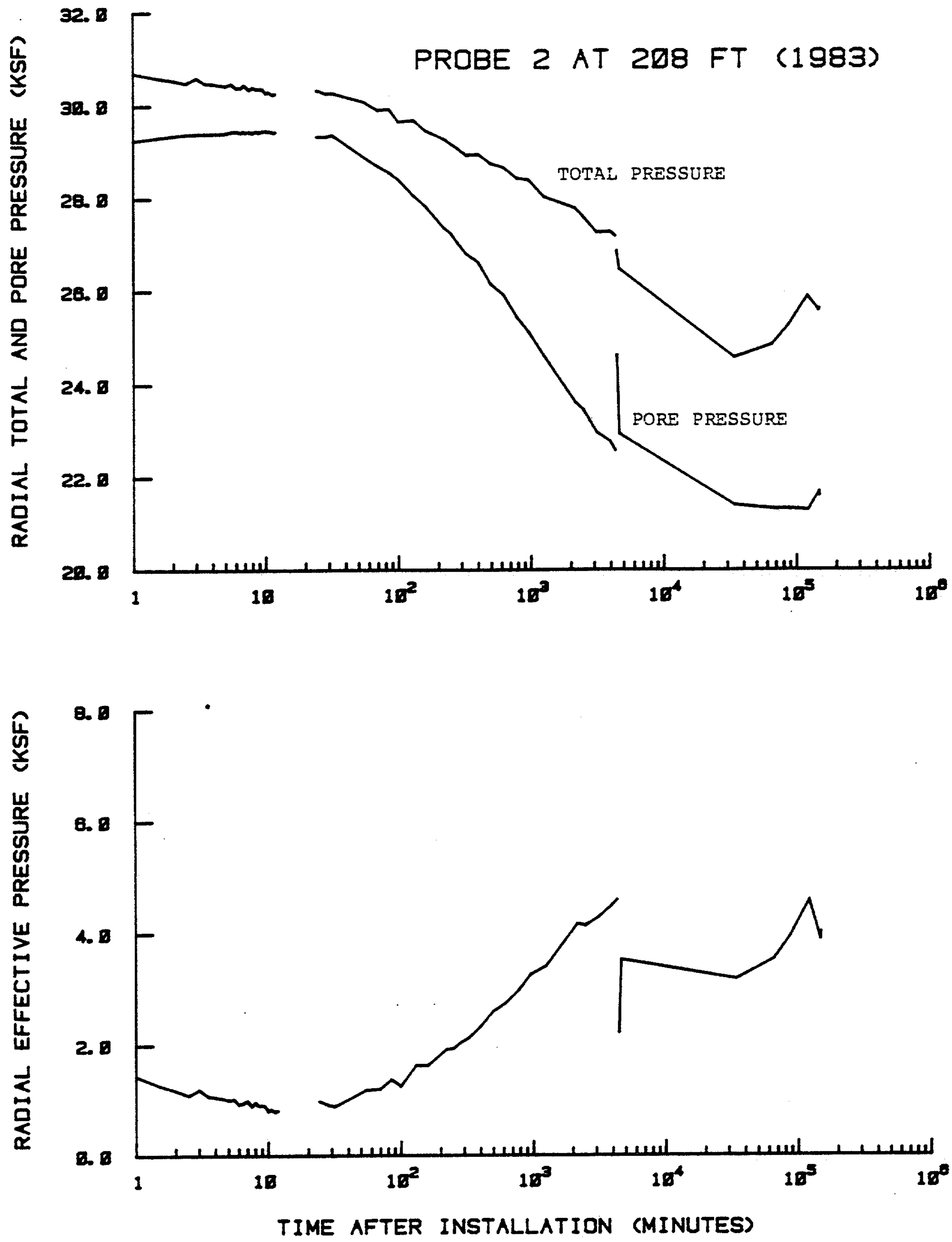
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 3 AT 208 FT - 8 HR TEST

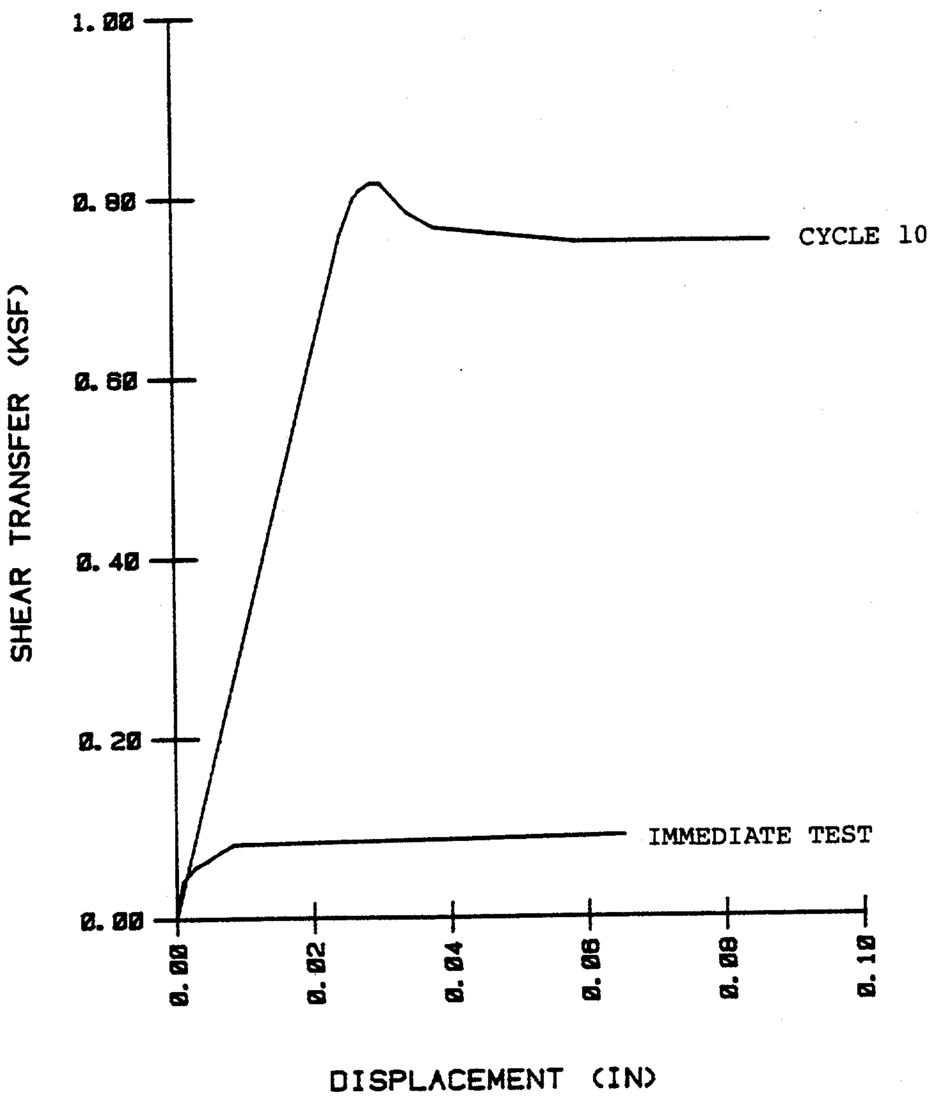
SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 4 AT 208 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 5 AT 208 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)

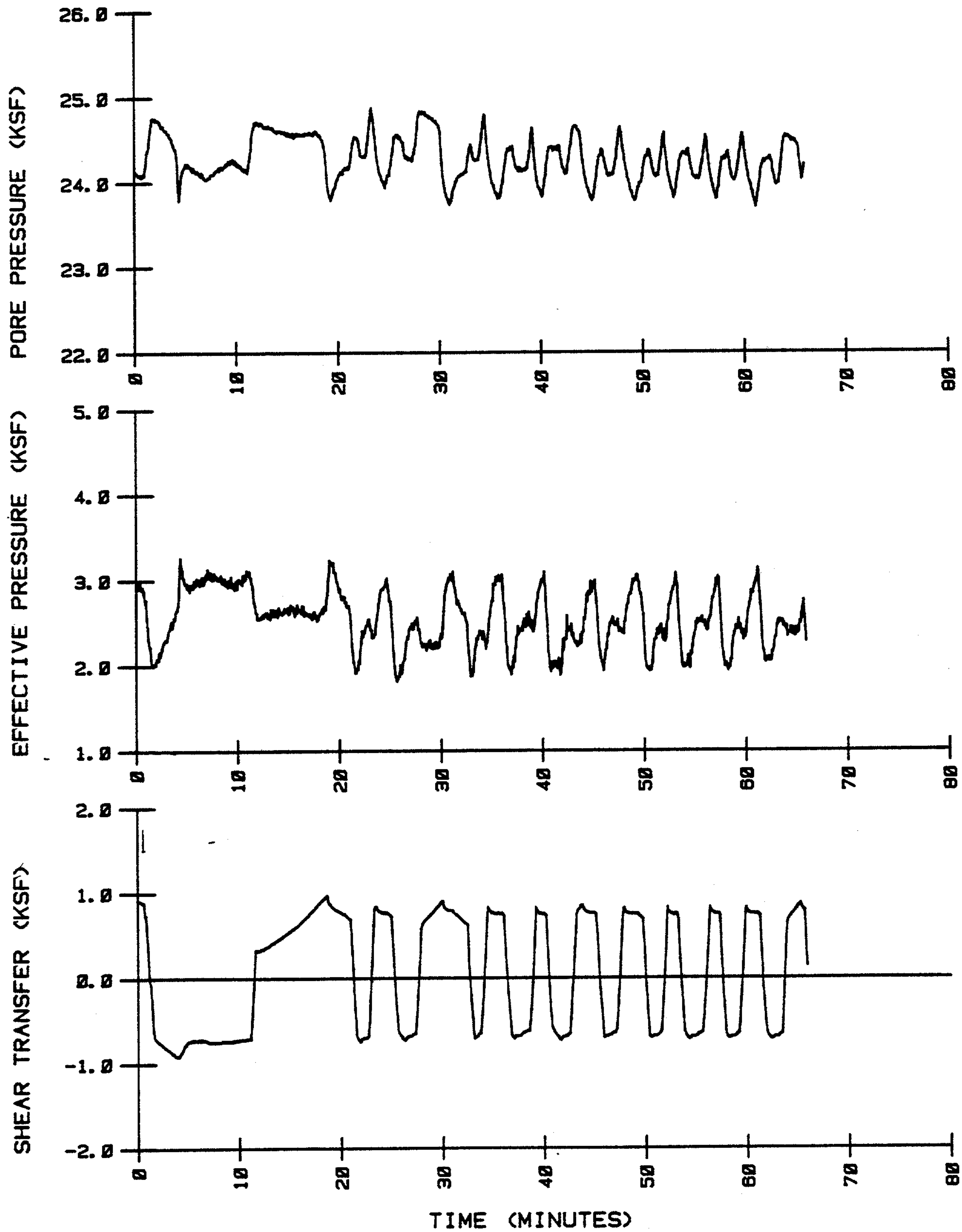


PROBE 2 AT 208 FT - 72 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 5 AT 208 FEET

PLATE 5.85

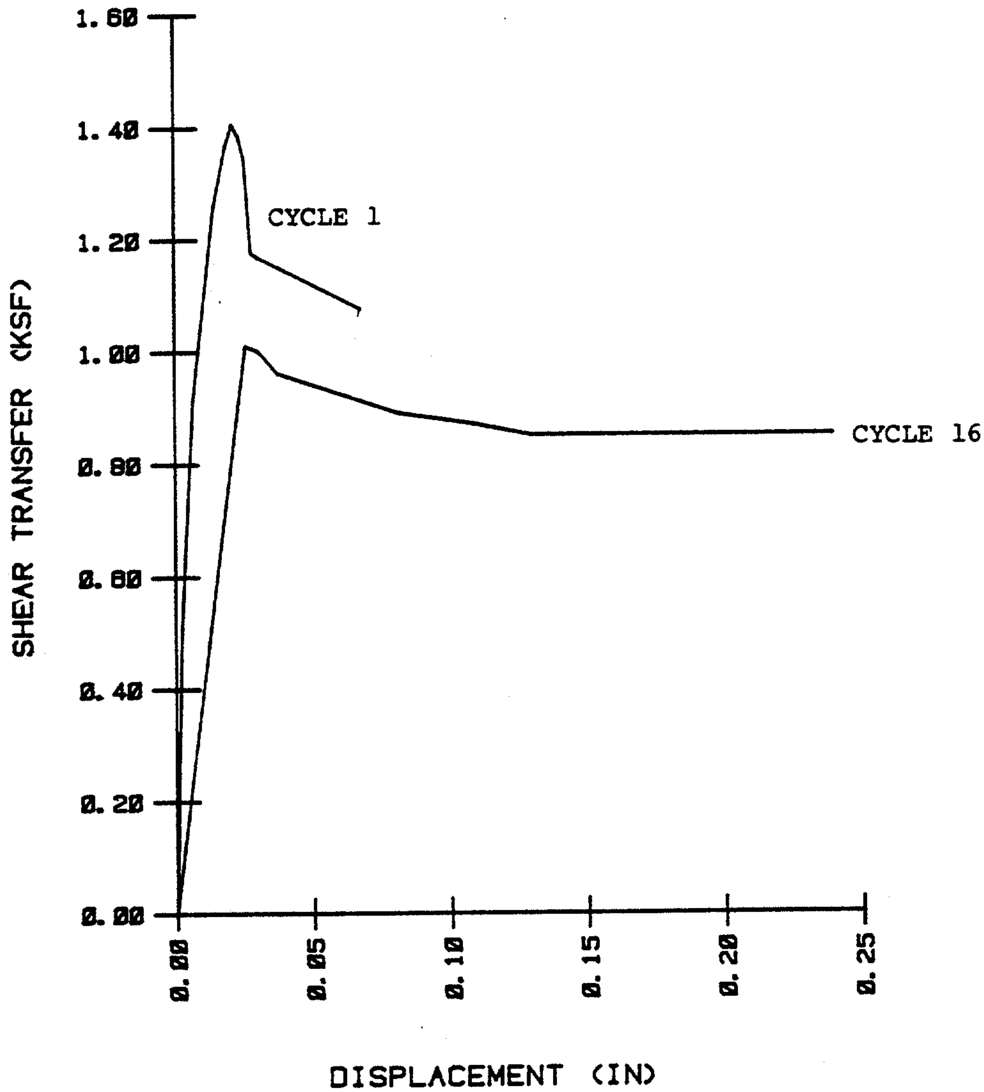
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 2 AT 208 FT - 72 HR TEST

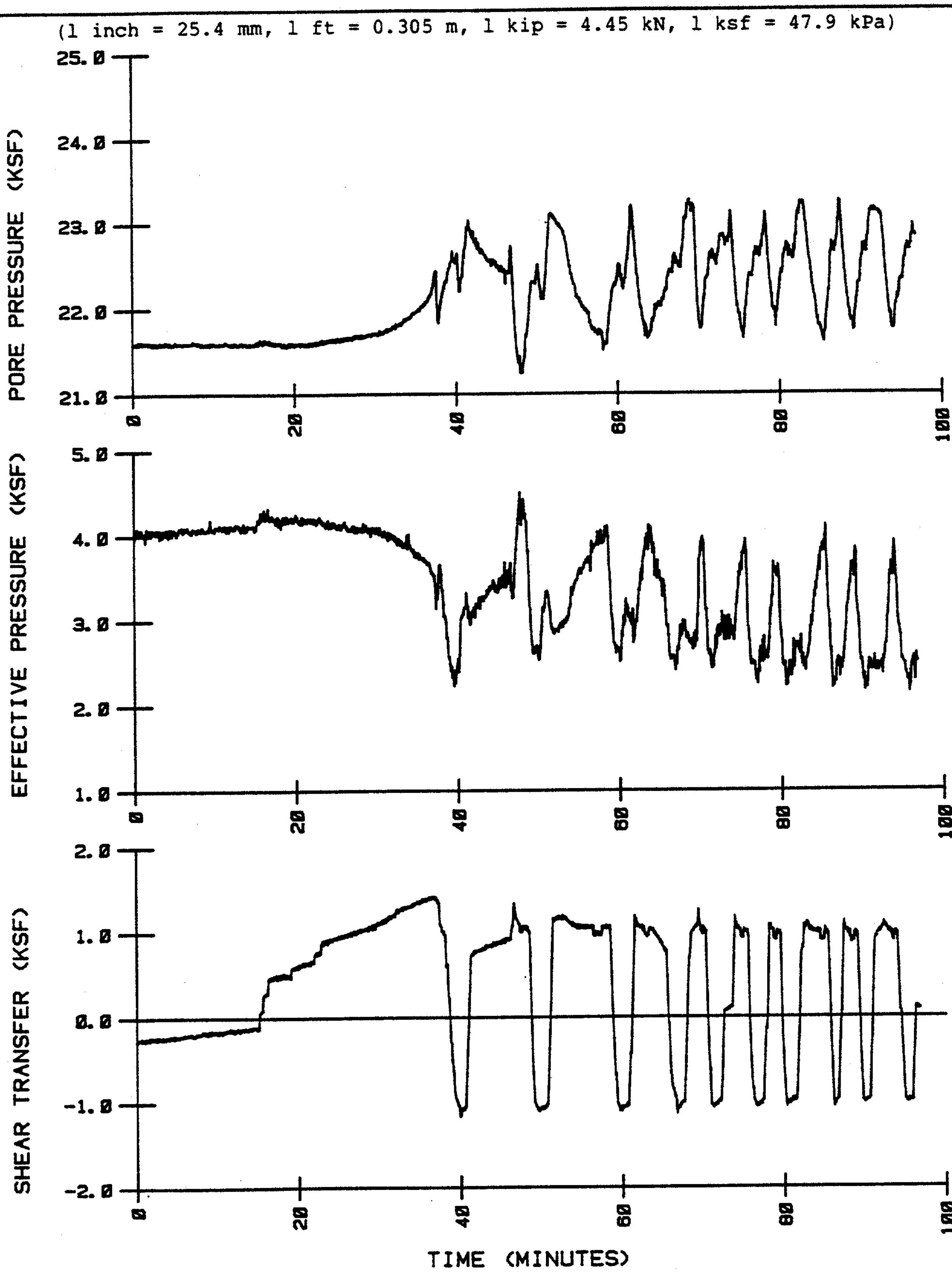
SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 5 AT 208 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



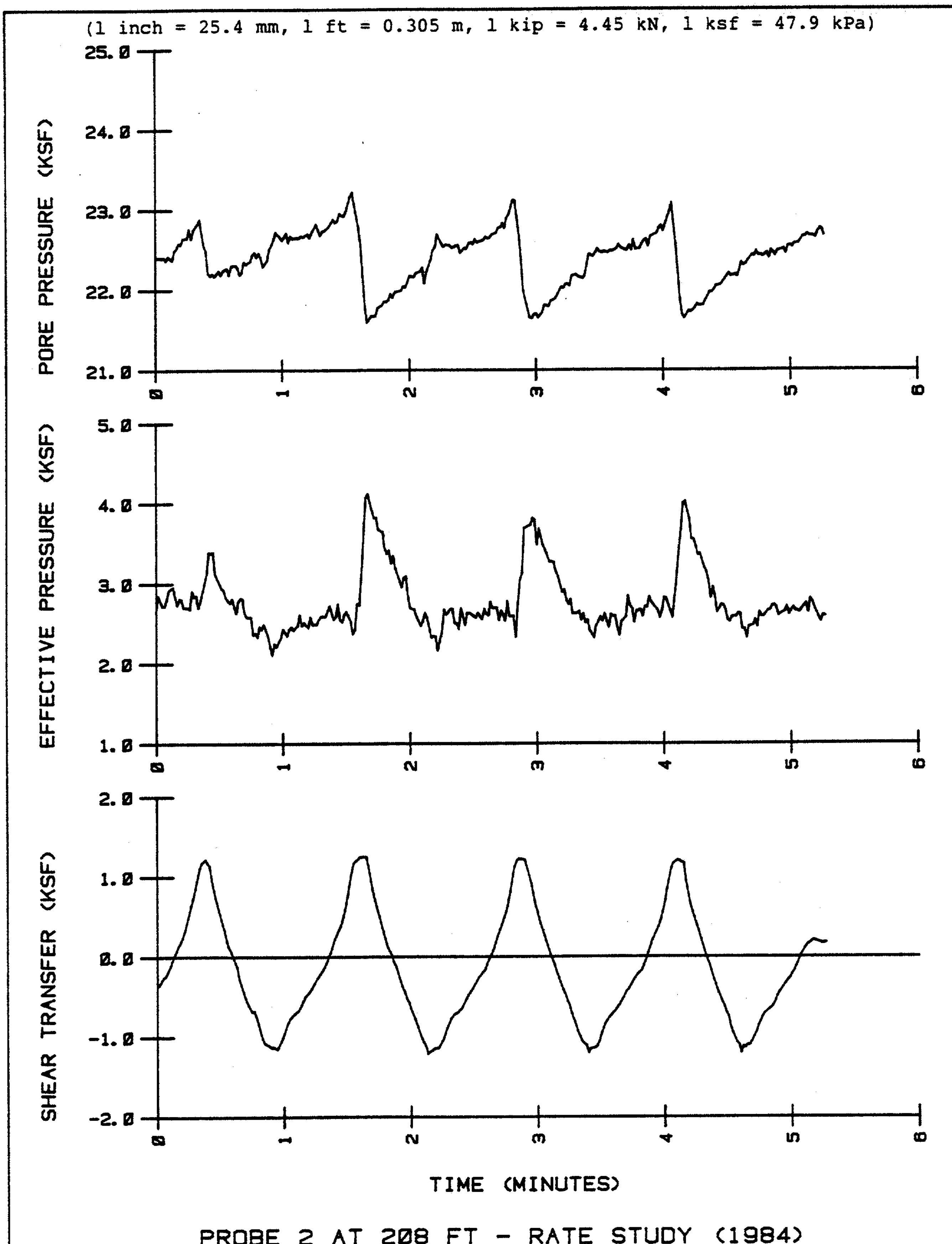
PROBE 2 AT 208 FT - 2500 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 5 AT 208 FEET



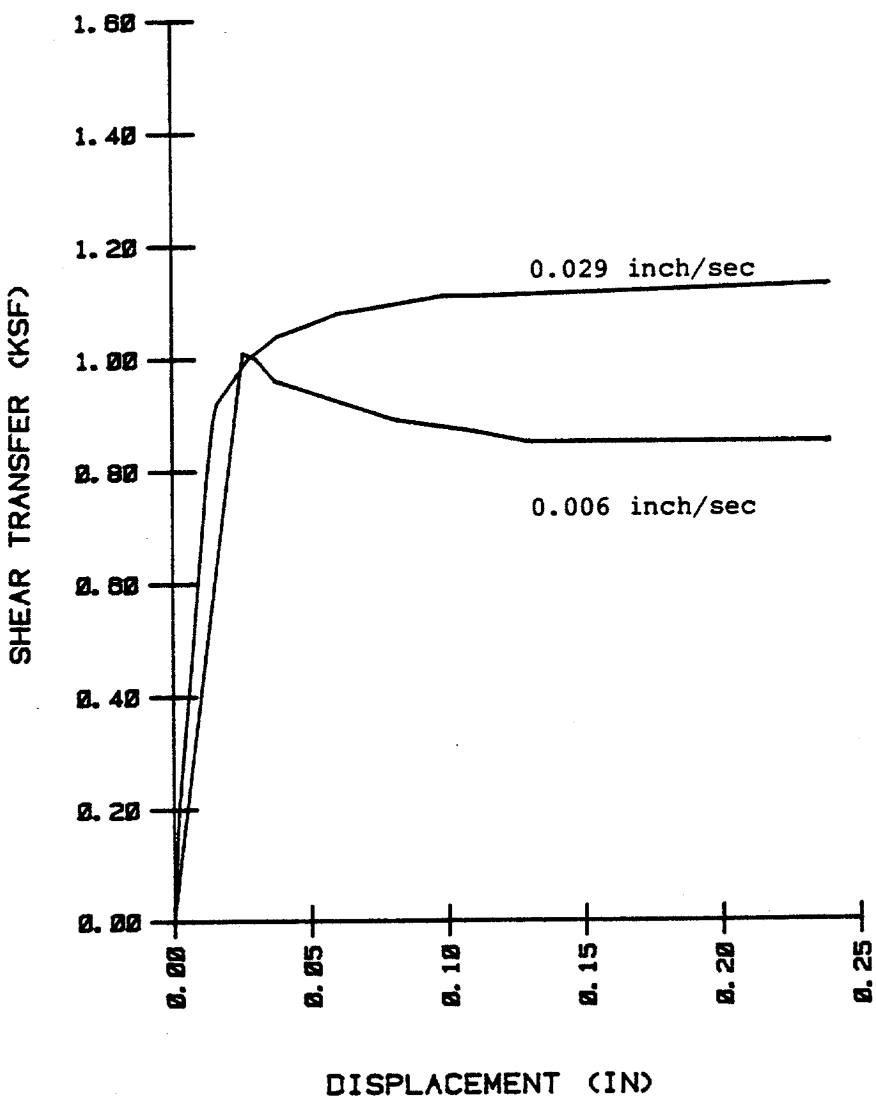
PROBE 2 AT 208 FT - 2500 HR TEST

SOIL BEHAVIOR RECORDED DURING THE SECOND CYCLIC TEST IN EXPERIMENT 5 AT 208 FEET



EFFECTS OF LOAD RATE ON SOIL BEHAVIOR RECORDED IN EXPERIMENT 5 AT 208 FEET

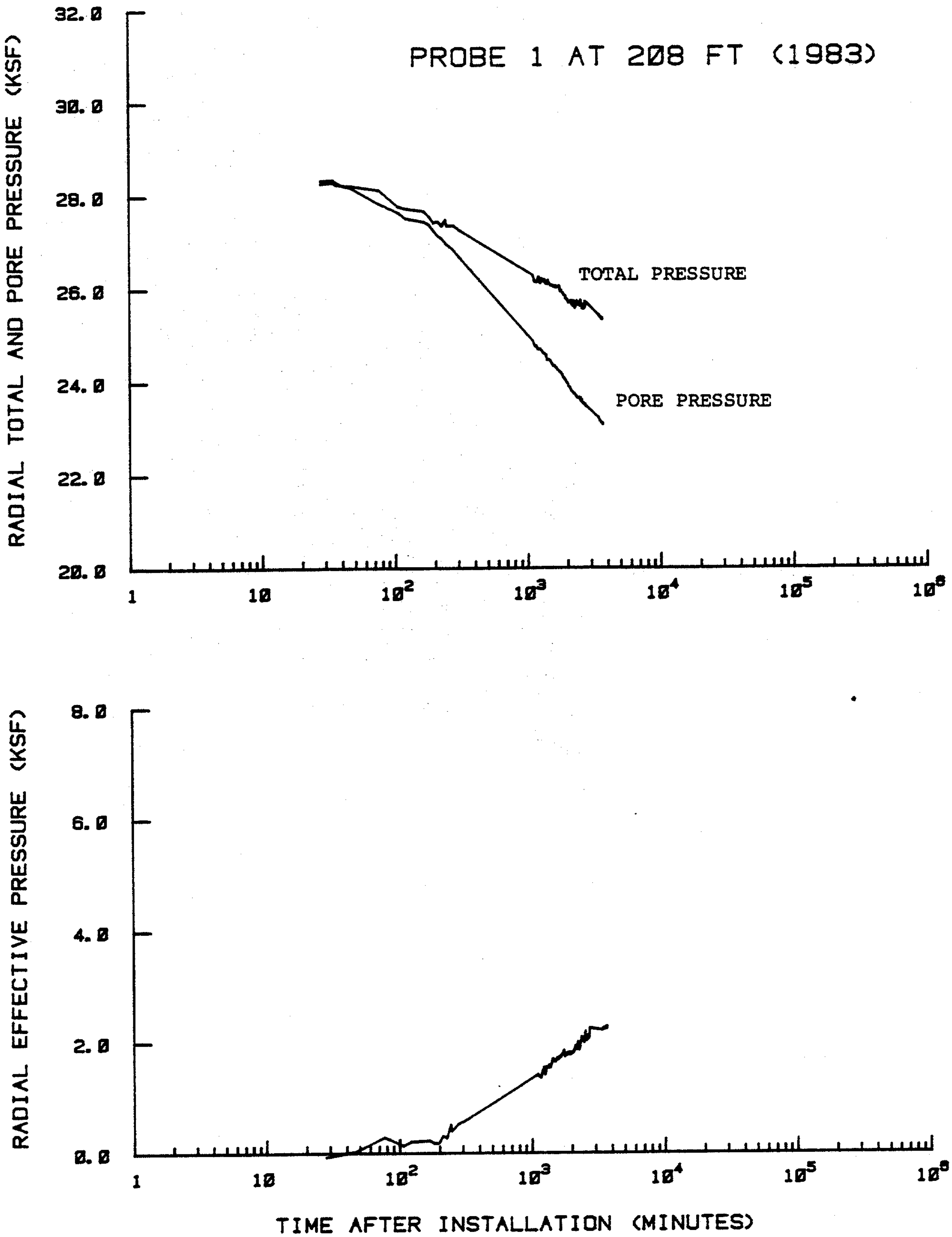
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 2 AT 208 FT - RATE STUDY

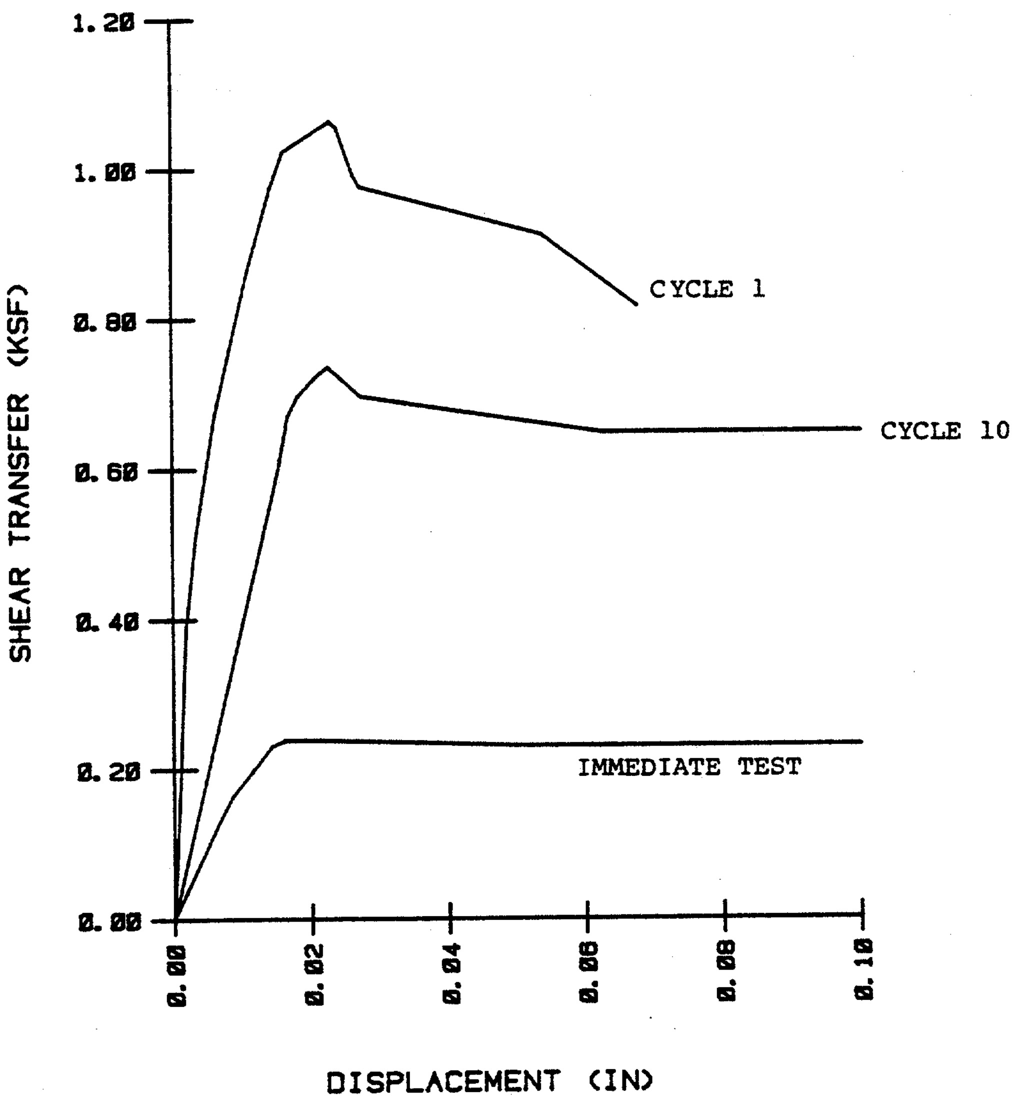
EFFECTS OF LOAD RATE ON THE SHEAR TRANSFER RECORDED IN EXPERIMENT 5 AT 208 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SOIL PRESSURES DURING CONSOLIDATION IN EXPERIMENT 6 AT 208 FEET

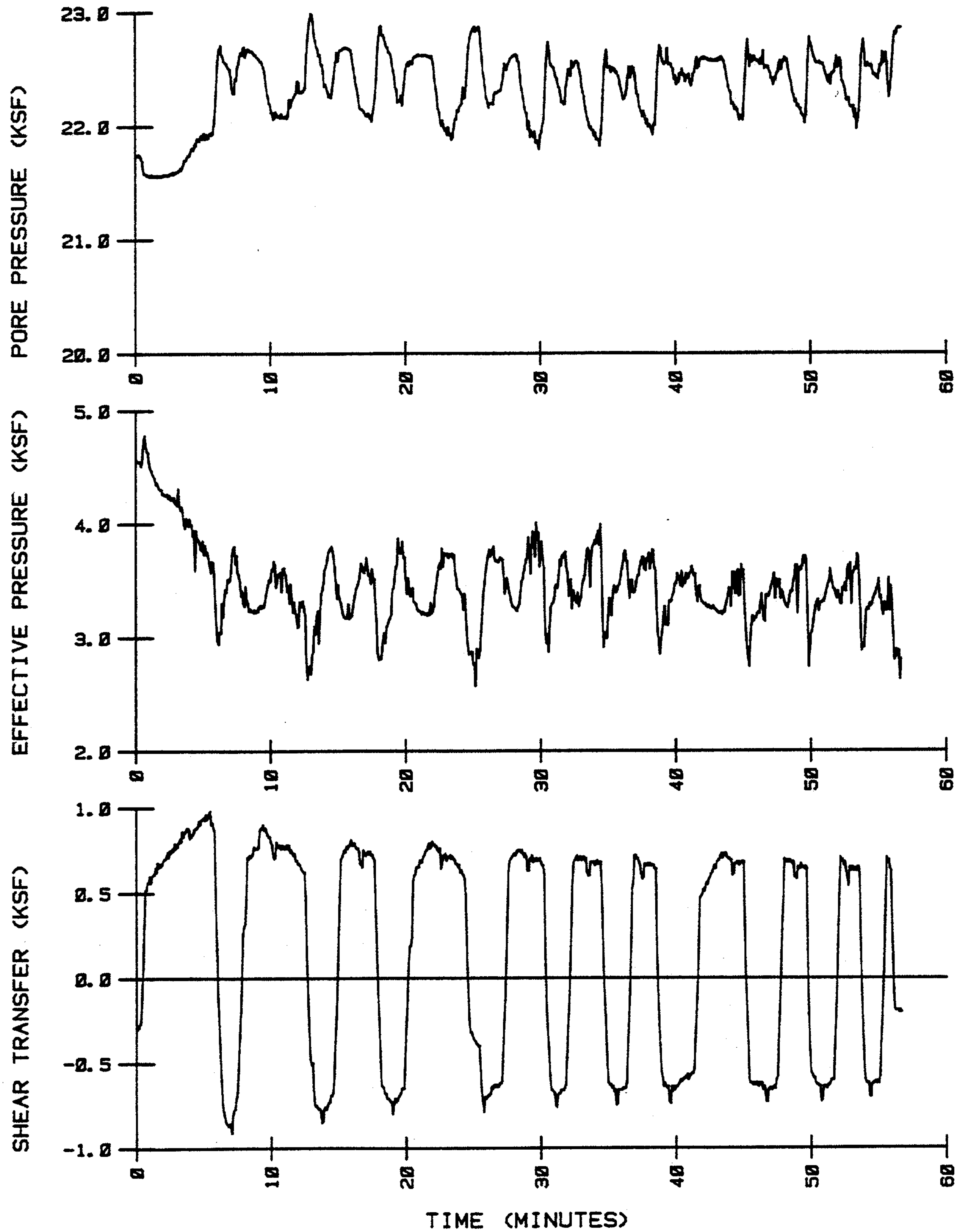
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 208 FT - 2500 HR TEST

SHEAR TRANSFER CURVES RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 6 AT 208 FEET

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)

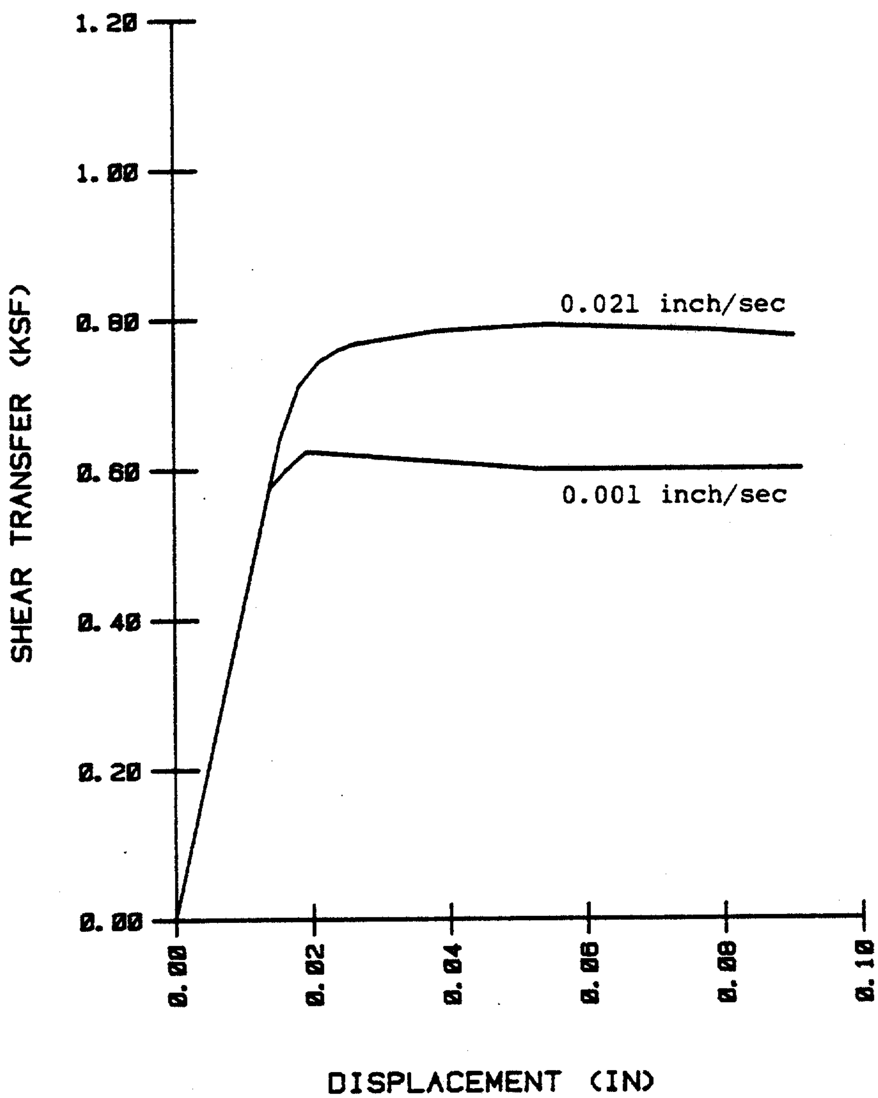


PROBE 1 AT 208 FT - 2500 HR TEST

SOIL BEHAVIOR RECORDED DURING THE INITIAL CYCLIC TEST IN EXPERIMENT 5 AT 208 FEET

PLATE 5.93

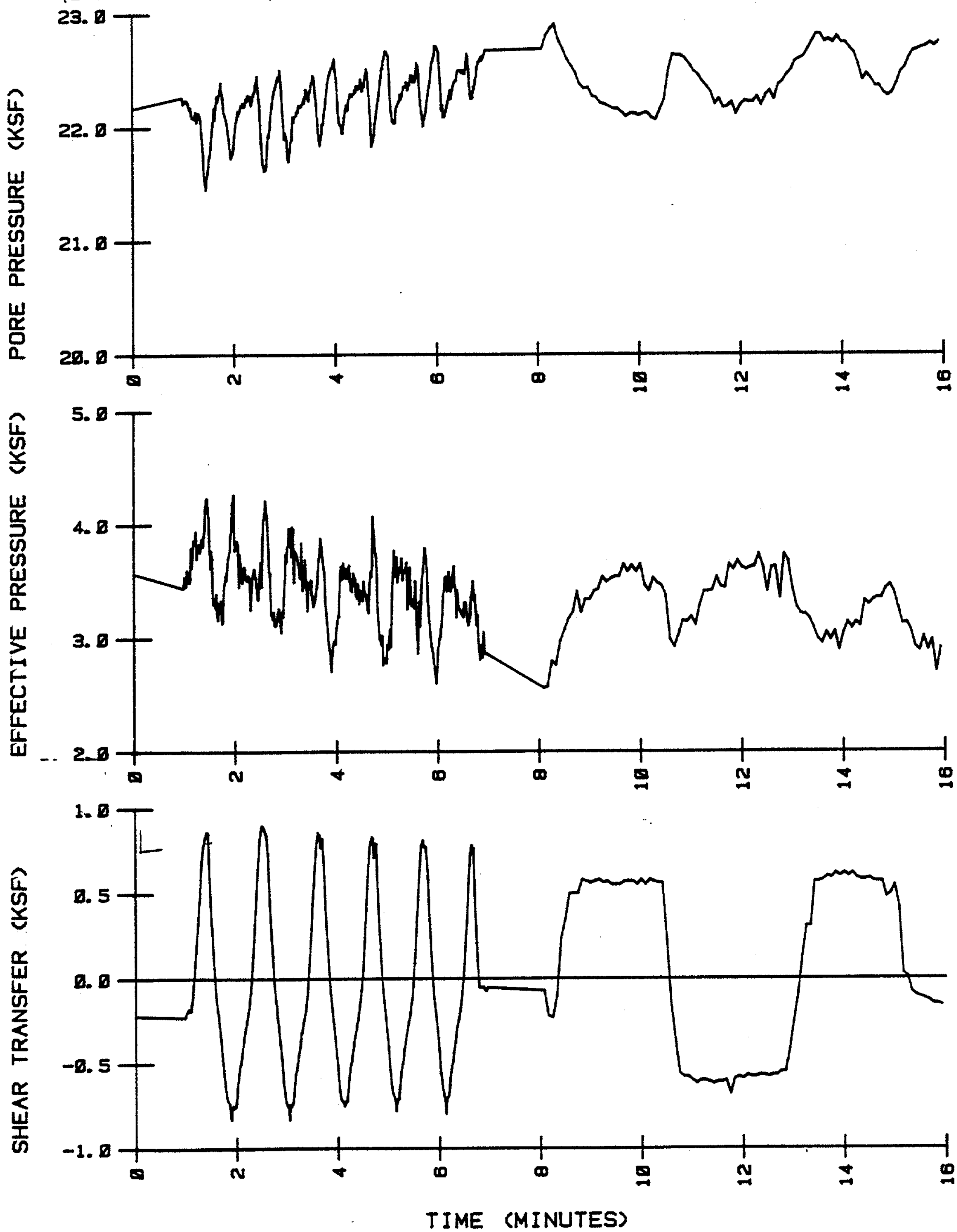
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 208 FT - RATE STUDY

EFFECTS OF LOAD RATE ON THE SHEAR TRANSFER RECORDED IN EXPERIMENT 6 AT 208 FEET (DURING LOADING)

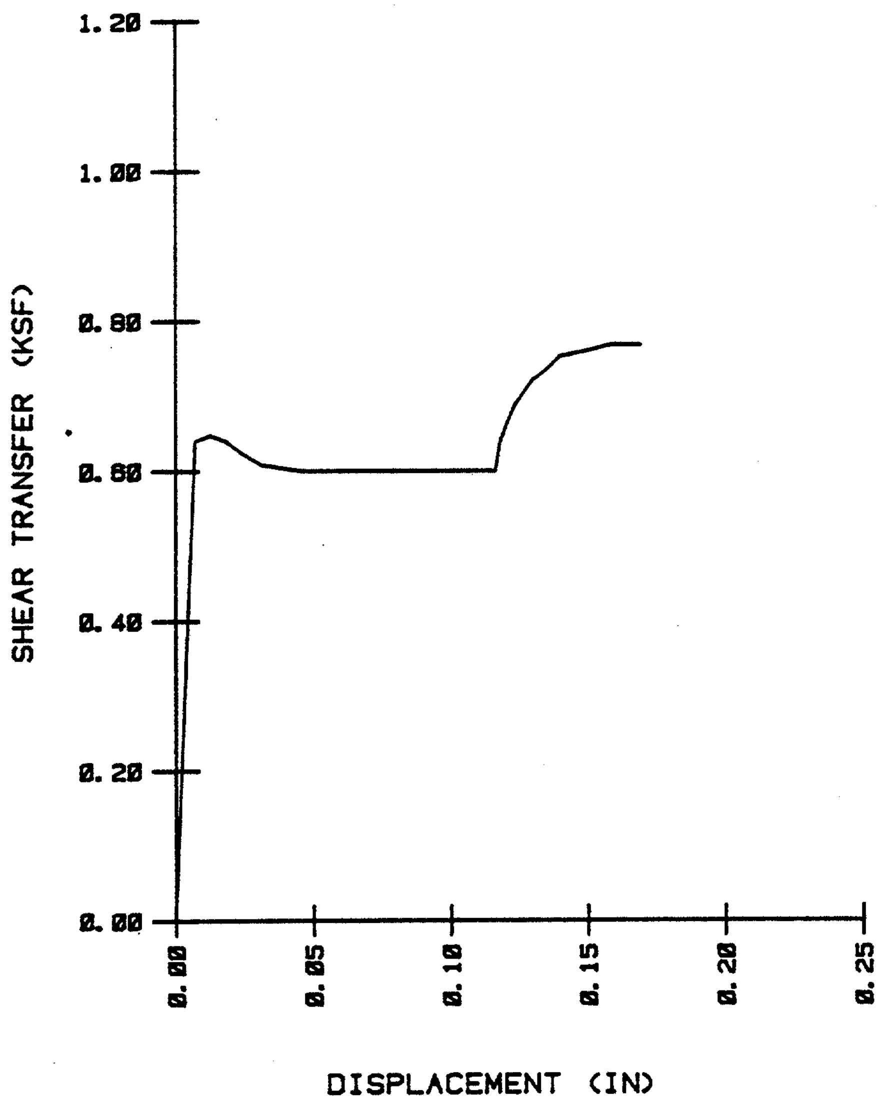
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 208 FT - RATE STUDY

EFFECTS OF LOAD RATE ON THE SOIL BEHAVIOR RECORDED IN EXPERIMENT 6 AT 208 FEET (DURING LOADING)

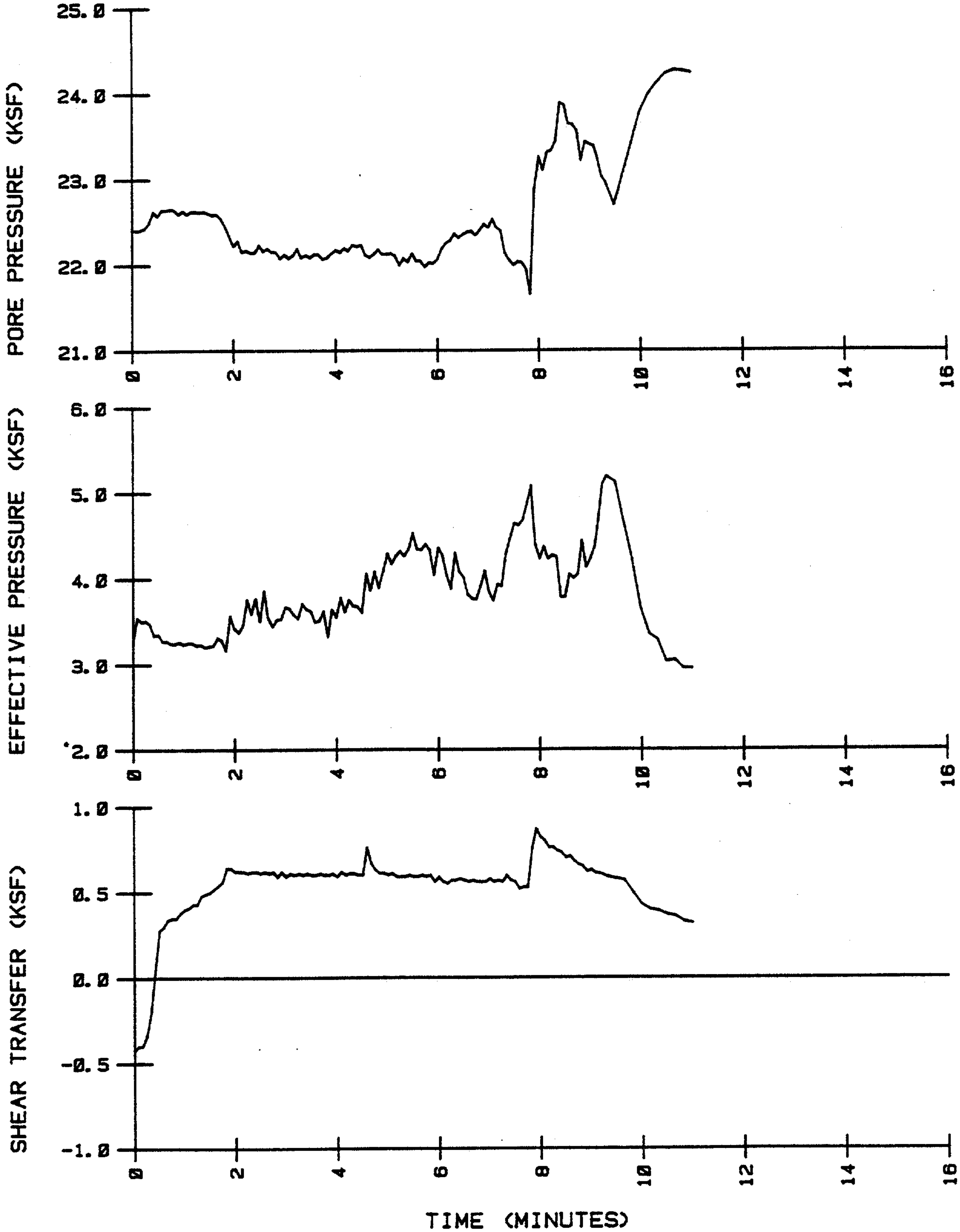
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



PROBE 1 AT 208 FT - PULL OUT TEST

EFFECTS OF LOAD RATE ON THE SHEAR TRANSFER RECORDED IN EXPERIMENT 6 AT 208 FEET (DURING PLASTIC SLIP)

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



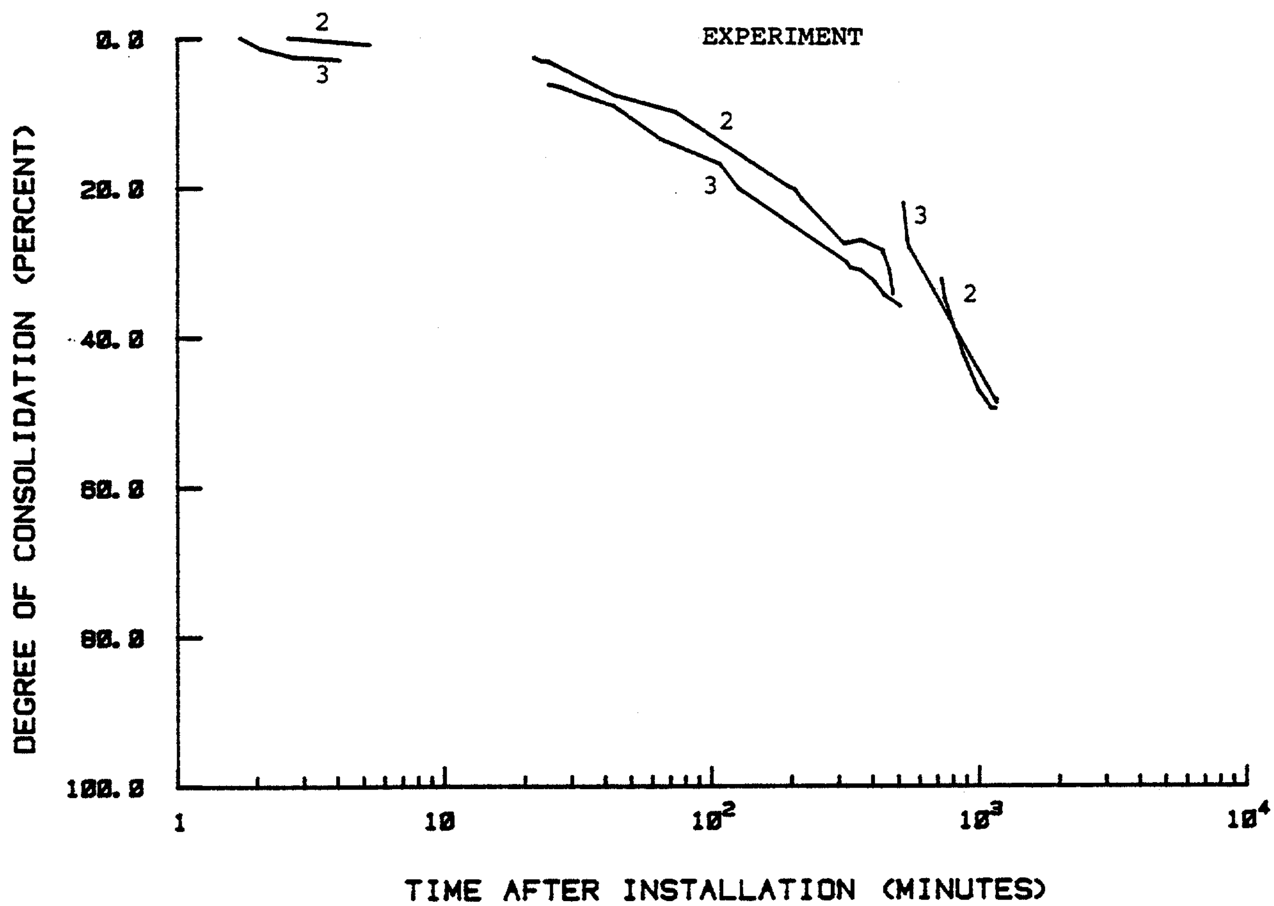
PROBE 1 AT 208 FT - PULL OUT TEST

EFFECTS OF LOAD RATE ON THE SOIL BEHAVIOR RECORDED
IN EXPERIMENT 6 AT 208 FEET (DURING PLASTIC SLIP)

APPENDIX 6

ILLUSTRATIONS FOR CHAPTER 6

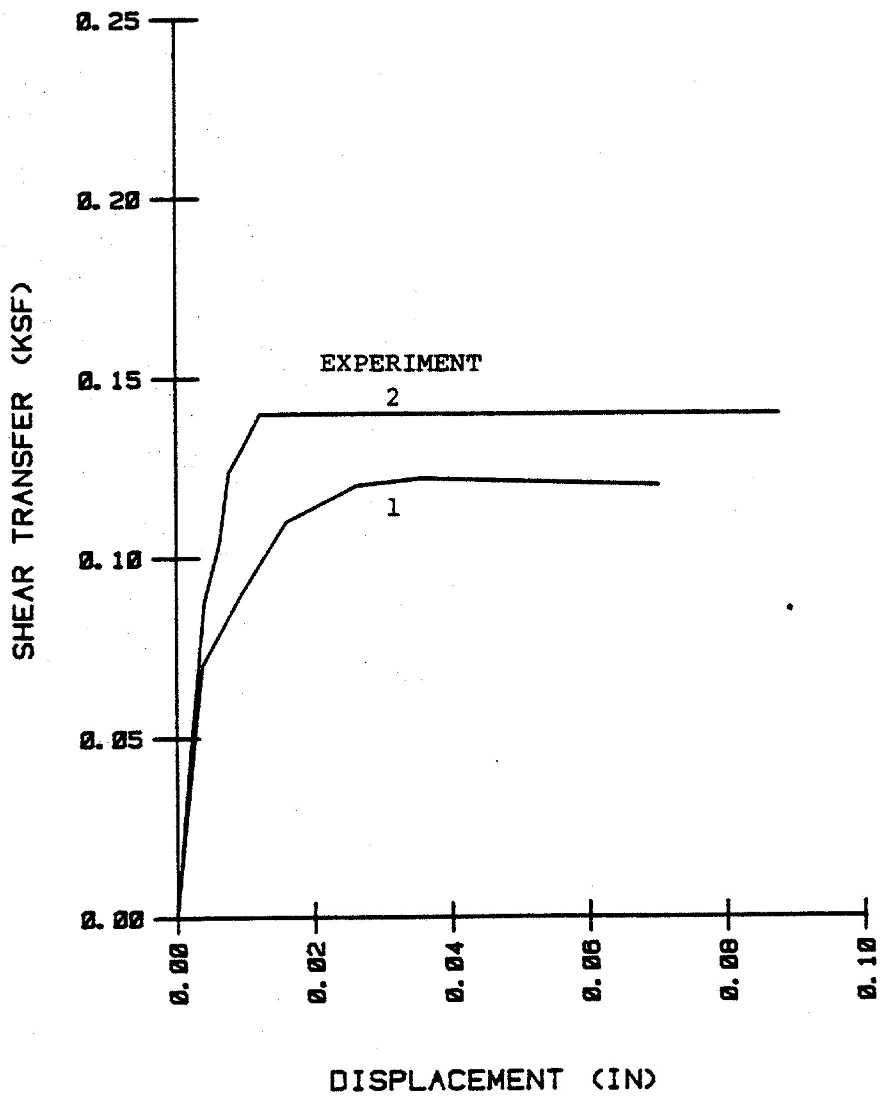
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



CONSOLIDATION AT 58 FEET

NORMALIZED PORE PRESSURE AT THE 58-FT DEPTH

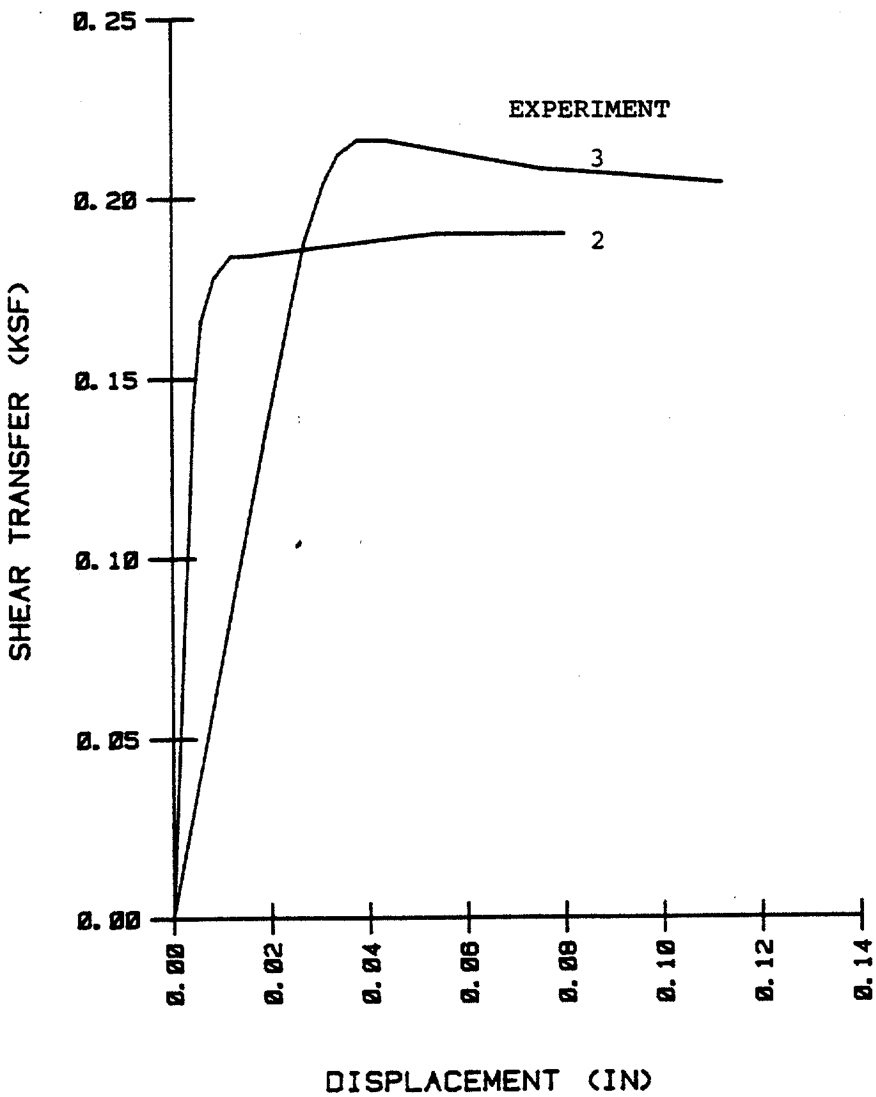
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



INITIAL TESTS AT 58 FEET

MAXIMUM SHEAR TRANSFER AFTER UNDISTURBED CONSOLIDATION

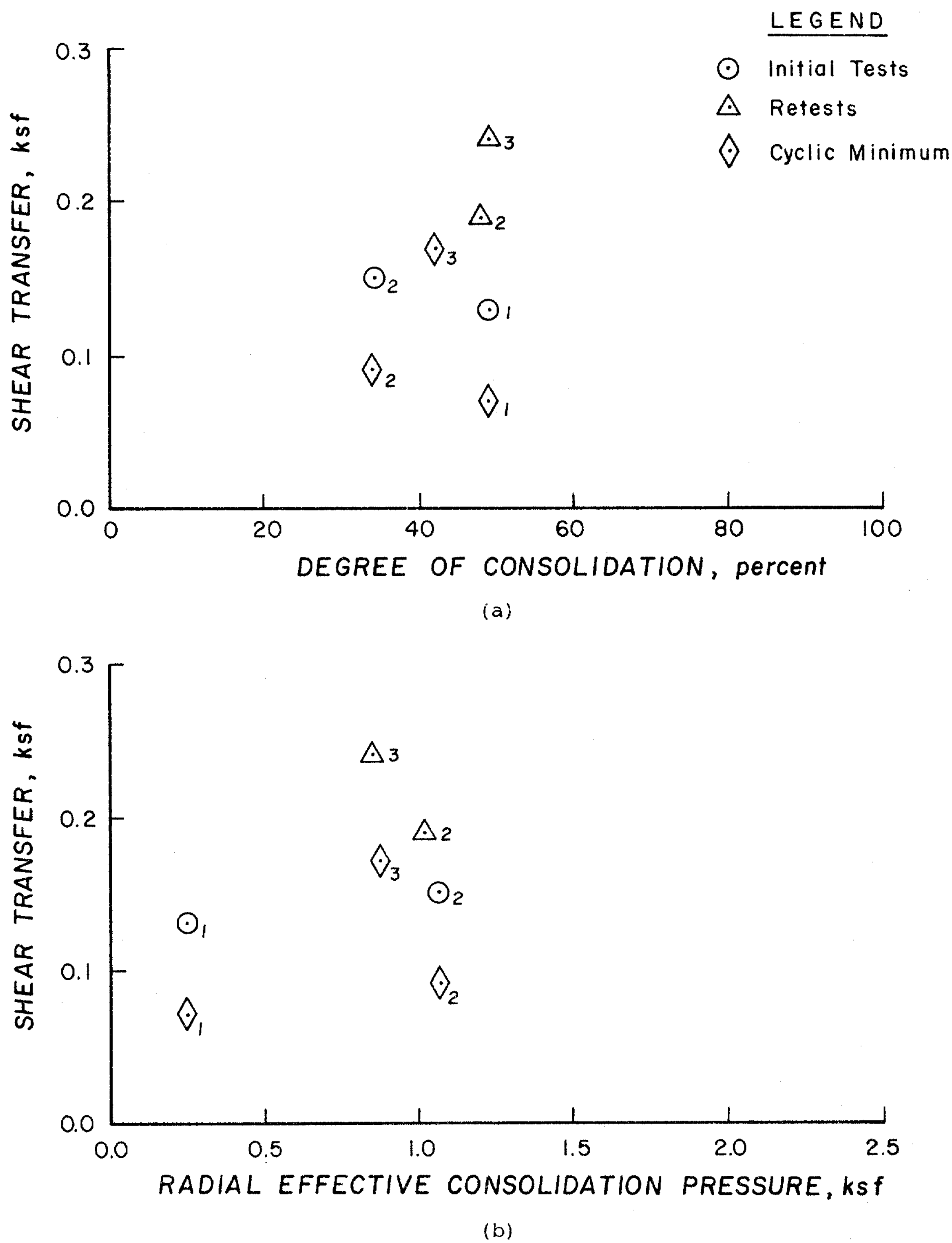
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



RETESTS AT 58 FEET

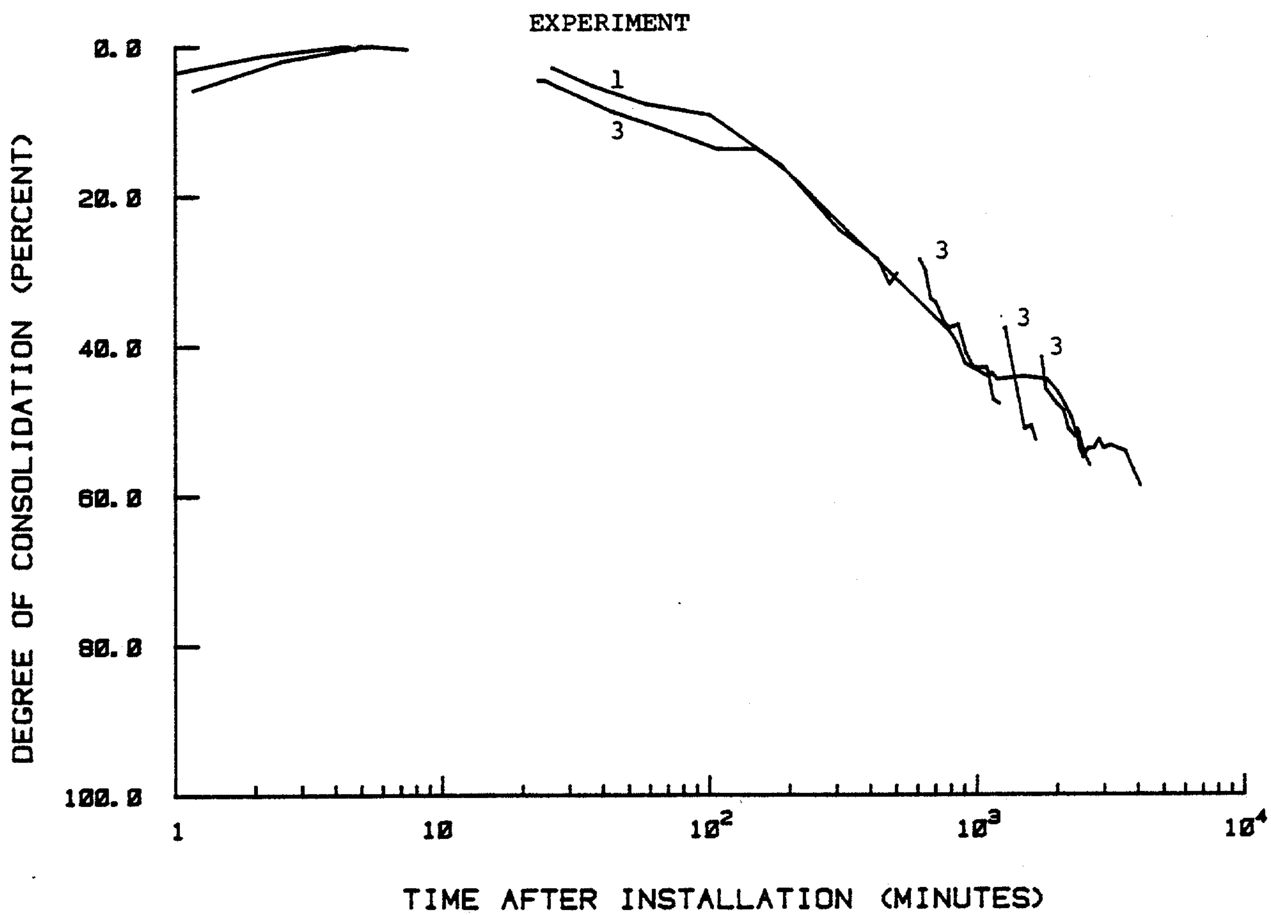
MAXIMUM SHEAR TRANSFER AFTER CYCLIC DEGRADATION AND RECONSOLIDATION

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SUMMARY OF SHEAR TRANSFER AND PRESSURE DATA AT THE 58-FT DEPTH

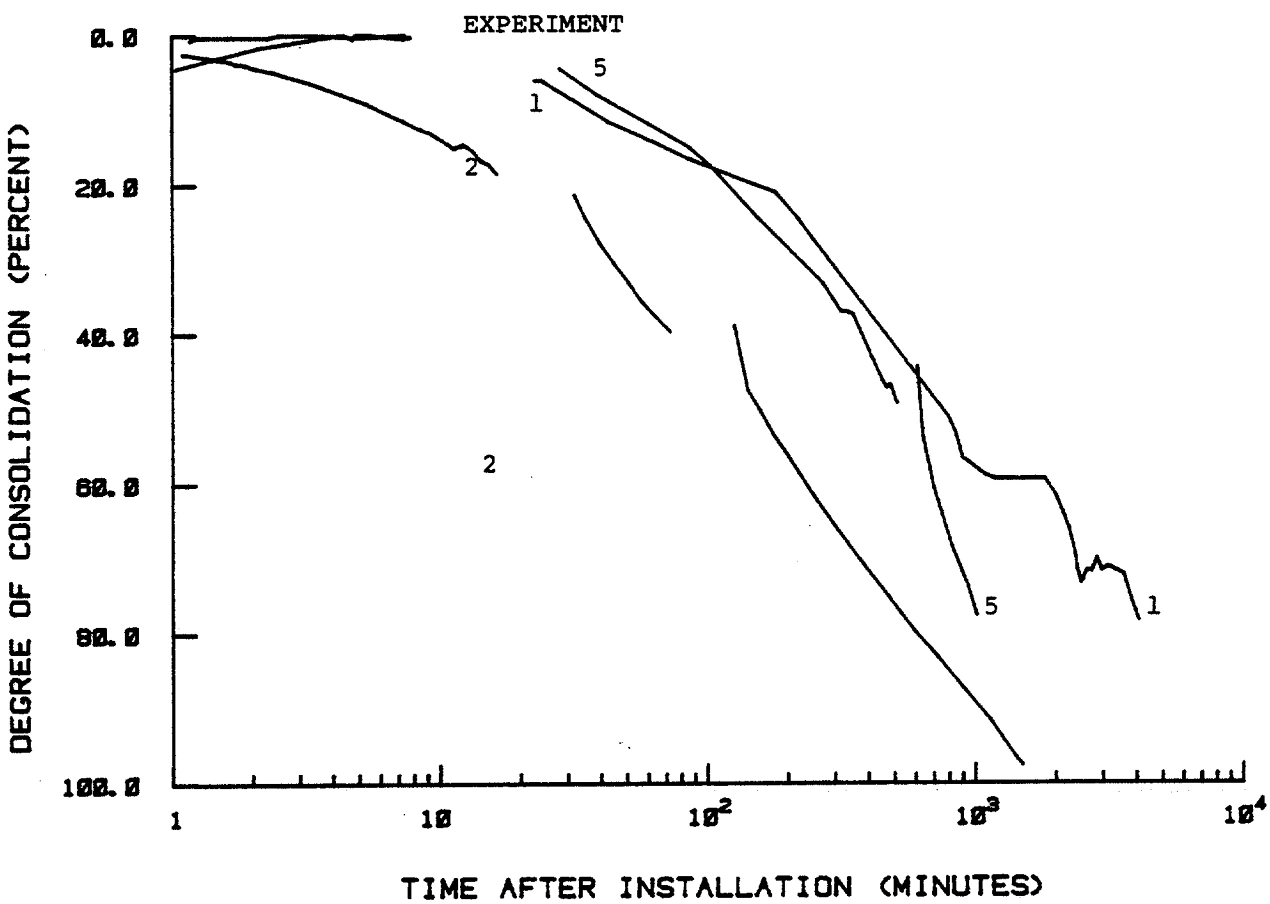
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



OPEN-END TESTS AT 148 FEET

NORMALIZED PORE PRESSURE RESPONSE AT THE 148-FT DEPTH

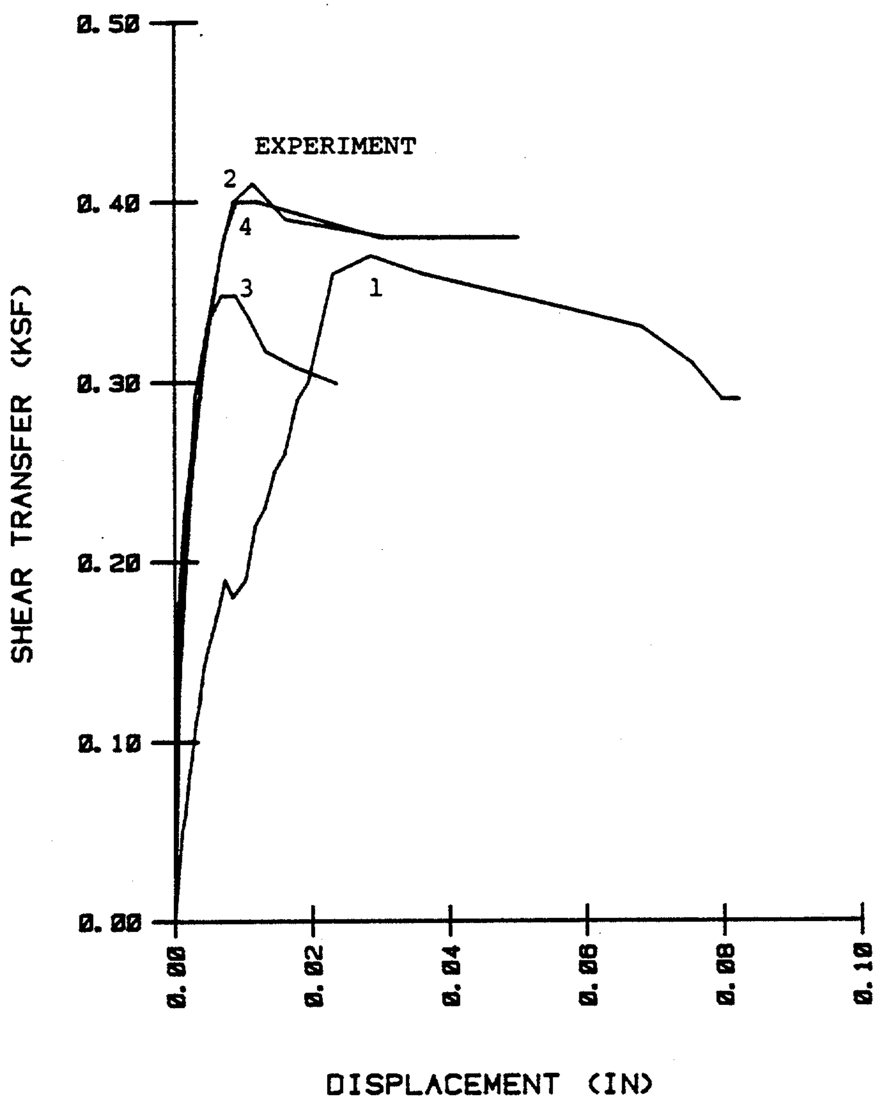
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



VARIED TIP CONDITIONS AT 148 FEET

NORMALIZED PORE PRESSURE RESPONSE AT THE 148-FT DEPTH

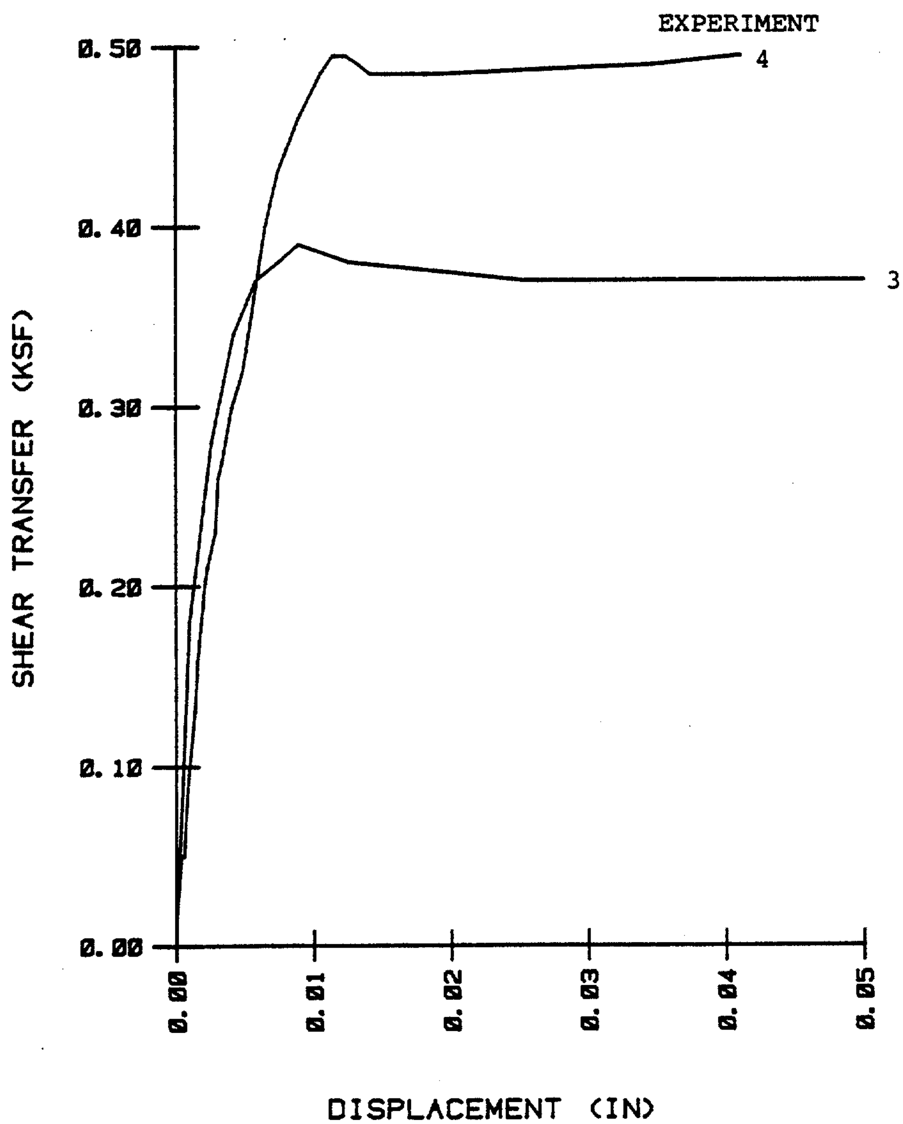
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



INITIAL TESTS AT 148 FEET

MAXIMUM SHEAR TRANSFER AFTER UNDISTURBED CONSOLIDATION

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)

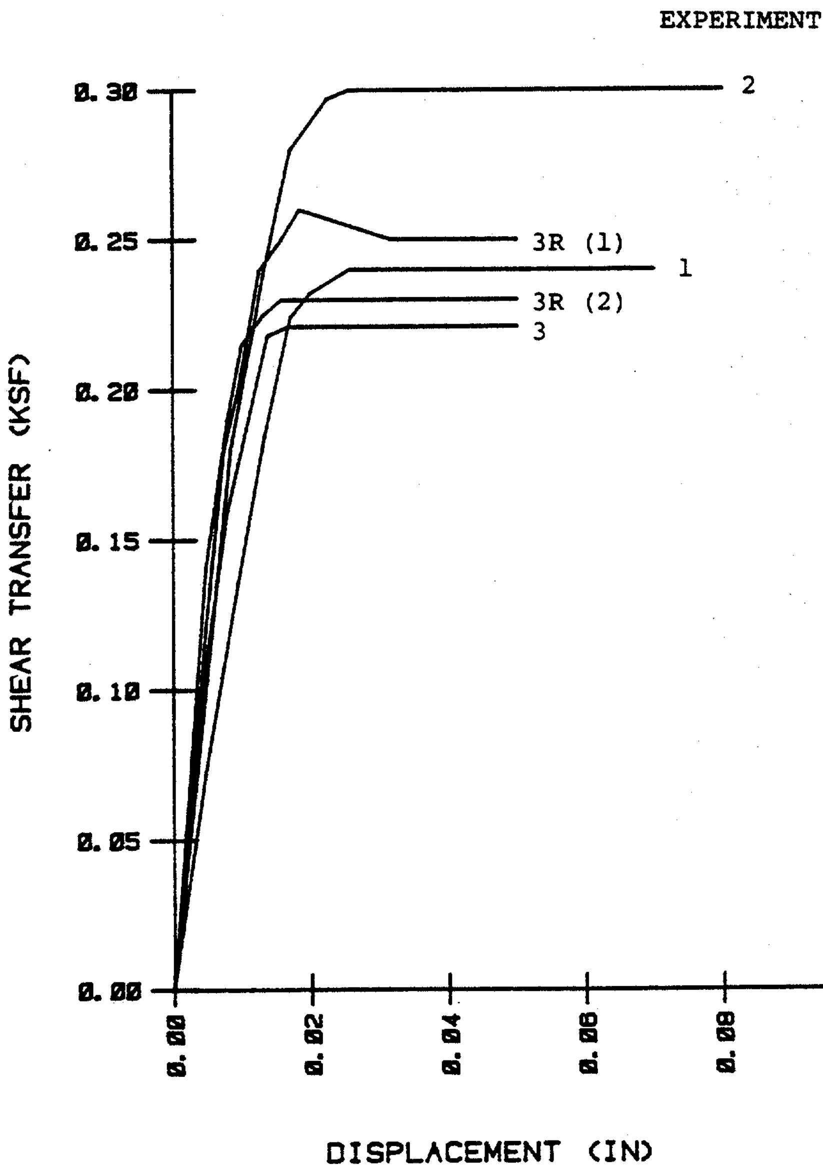


RETESTS AT 148 FEET

MAXIMUM SHEAR TRANSFER AFTER CYCLIC DEGRADATION AND RECONSOLIDATION

PLATE 6.8

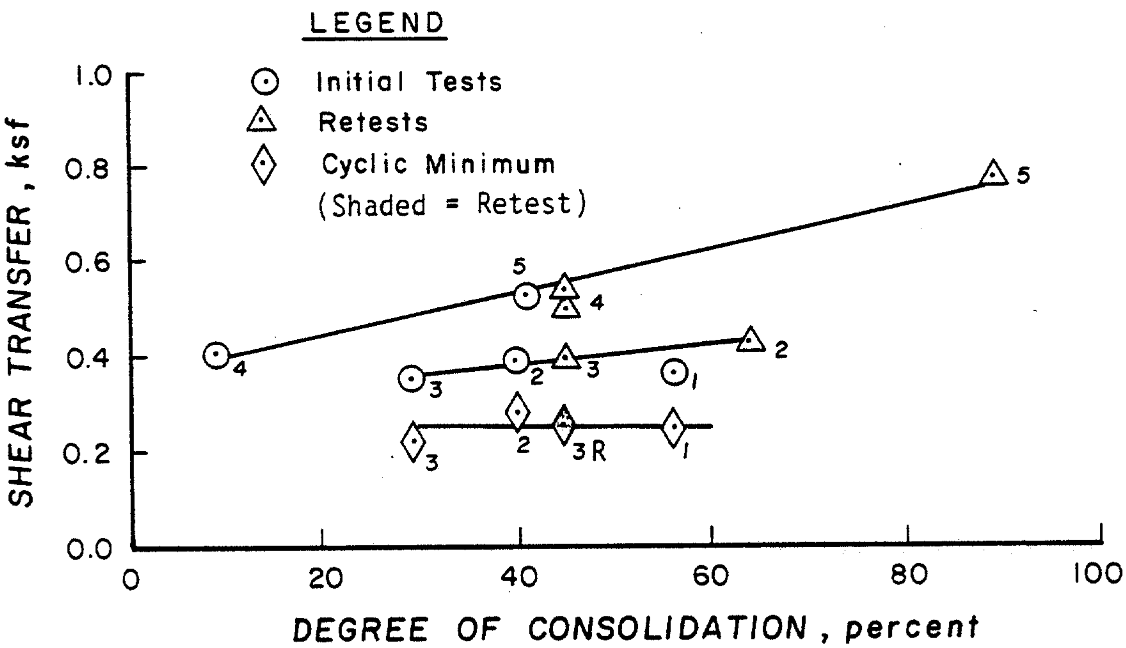
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



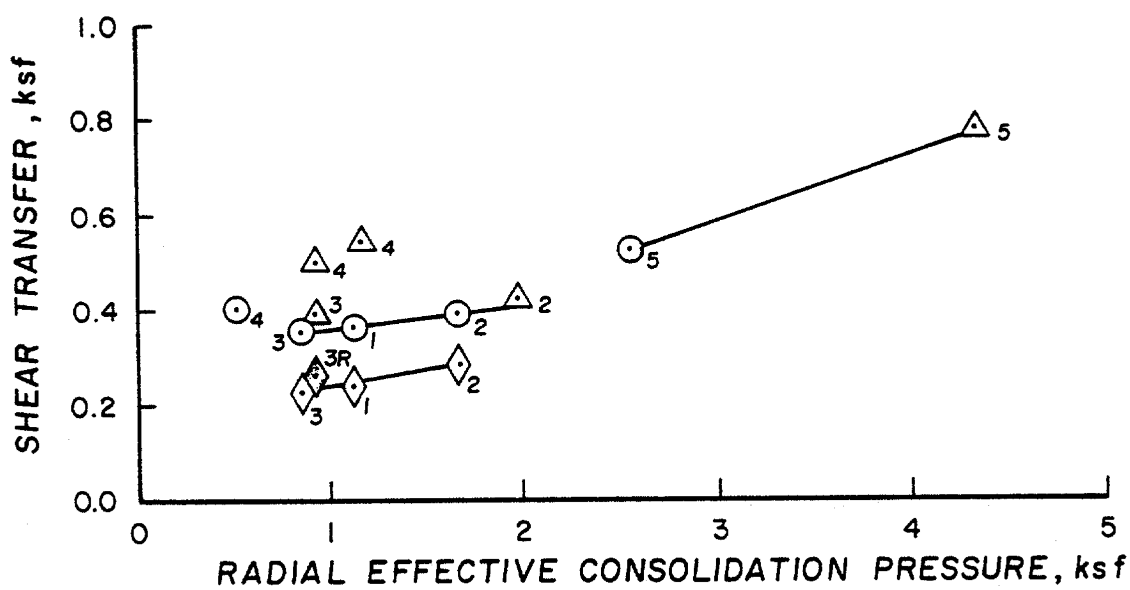
CYCLIC MINIMUM SHEAR TRANSFER AT 148 FT

LIMITING SHEAR TRANSFER AFTER CYCLIC DEGRADATION

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



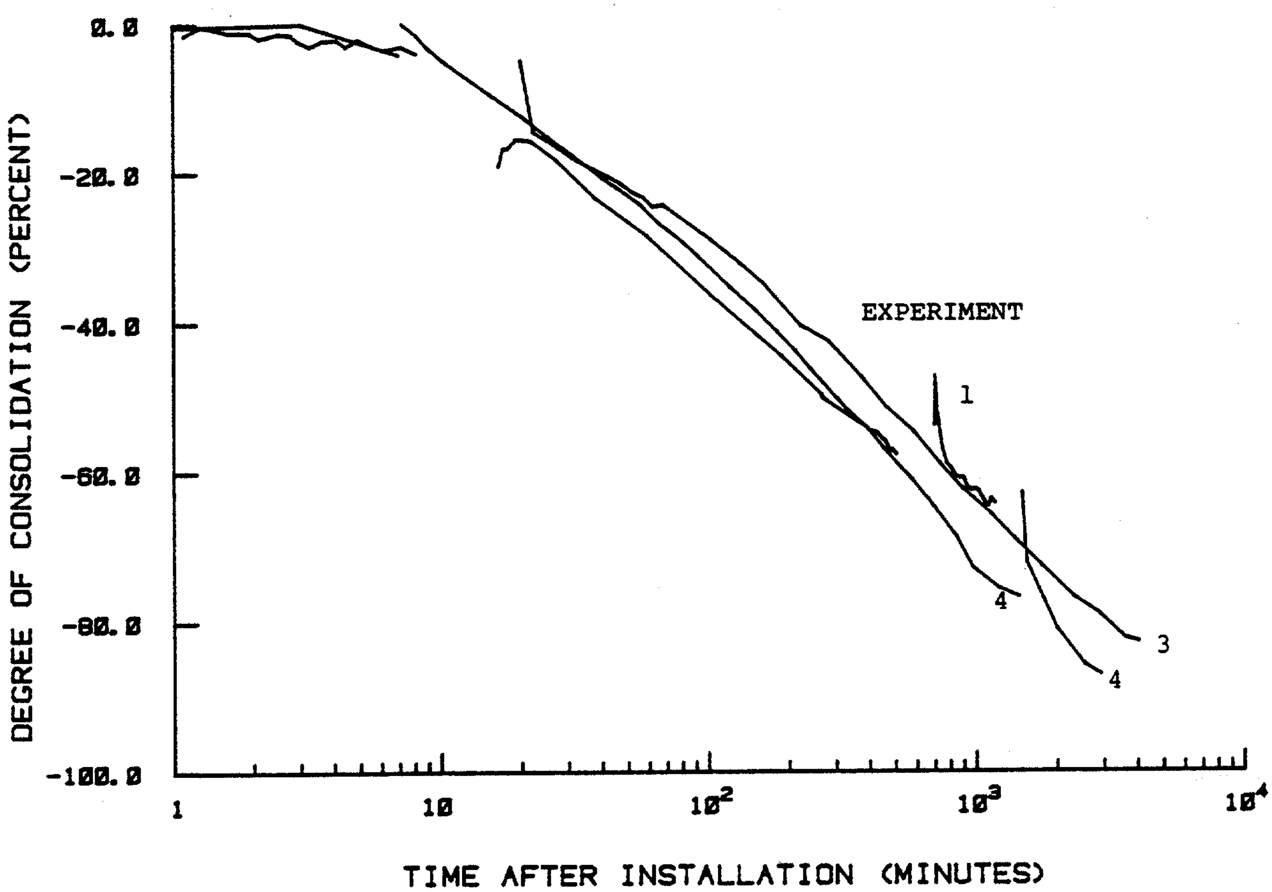
(a)



(b)

SUMMARY OF SHEAR TRANSFER AND PRESSURE DATA AT THE 148-FT DEPTH

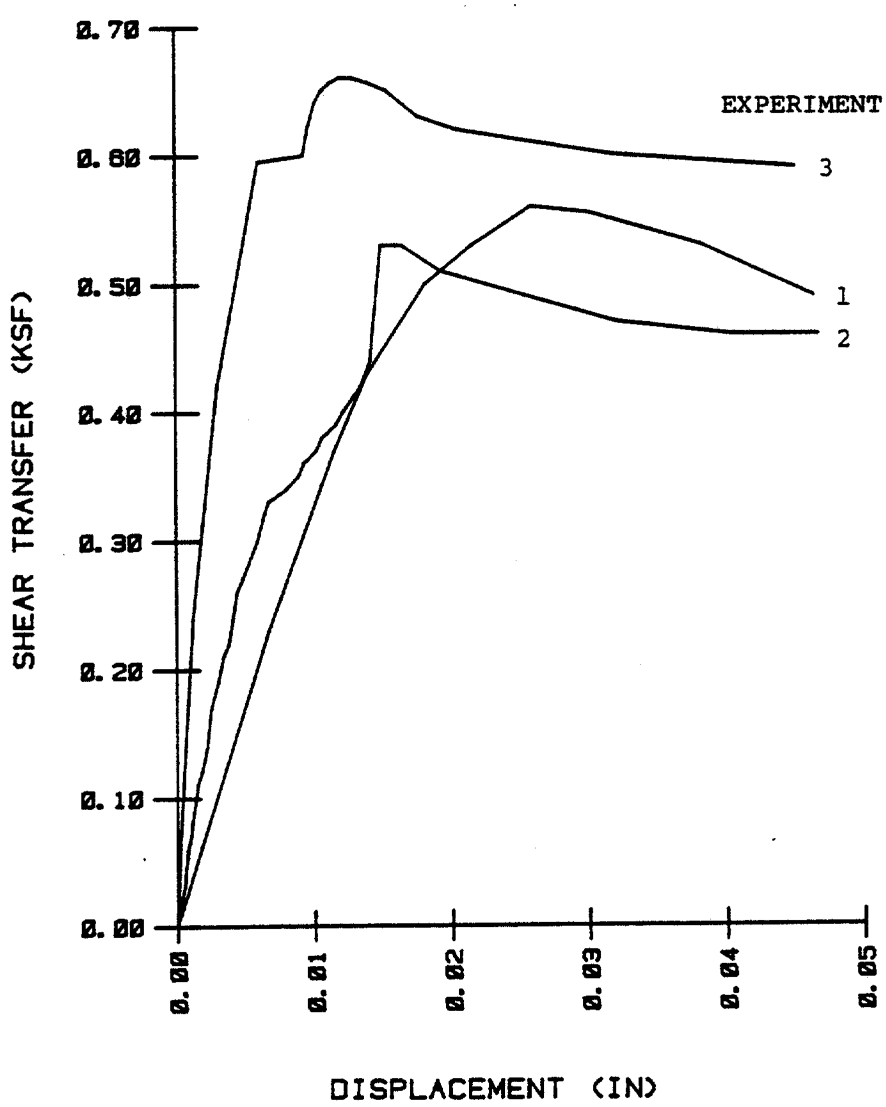
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



CONSOLIDATION AT 178 FEET

NORMALIZED PORE PRESSURE RESPONSE AT THE 178-FT DEPTH

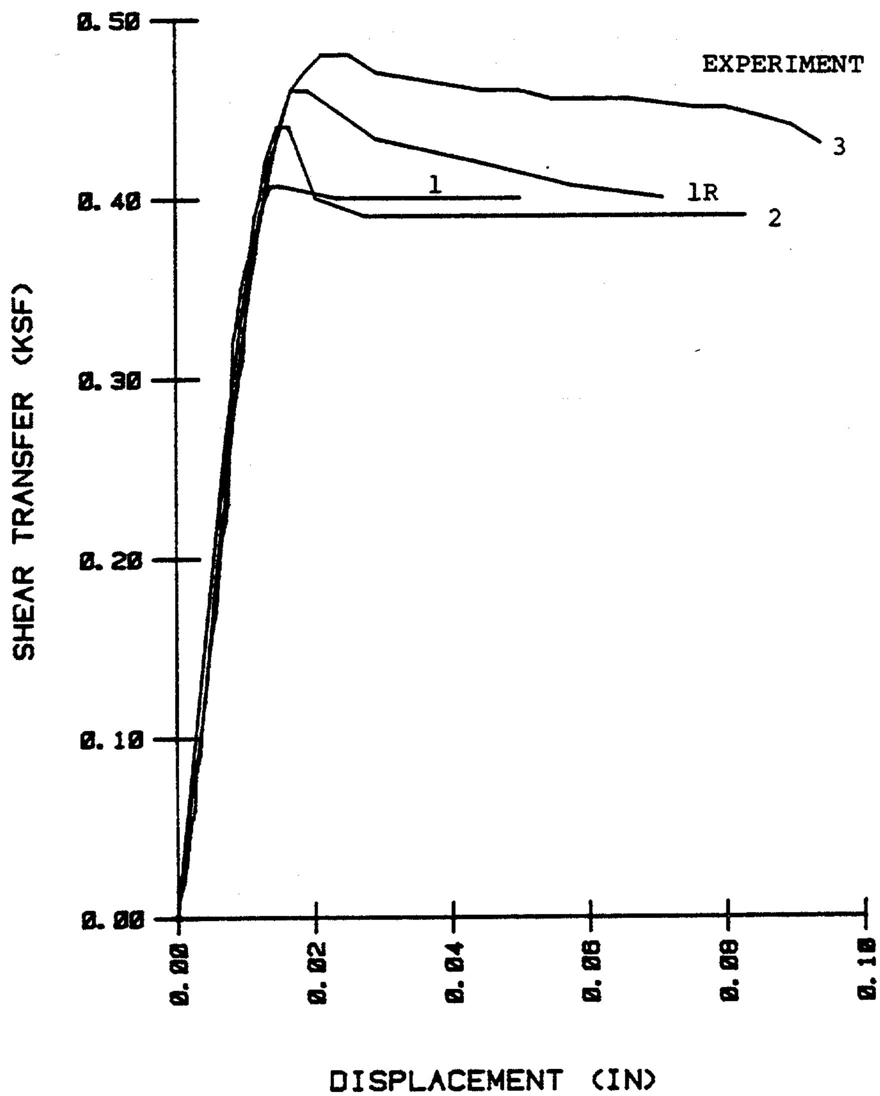
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



INITIAL TESTS AT 178 FEET

MAXIMUM SHEAR TRANSFER AFTER UNDISTURBED CONSOLIDATION

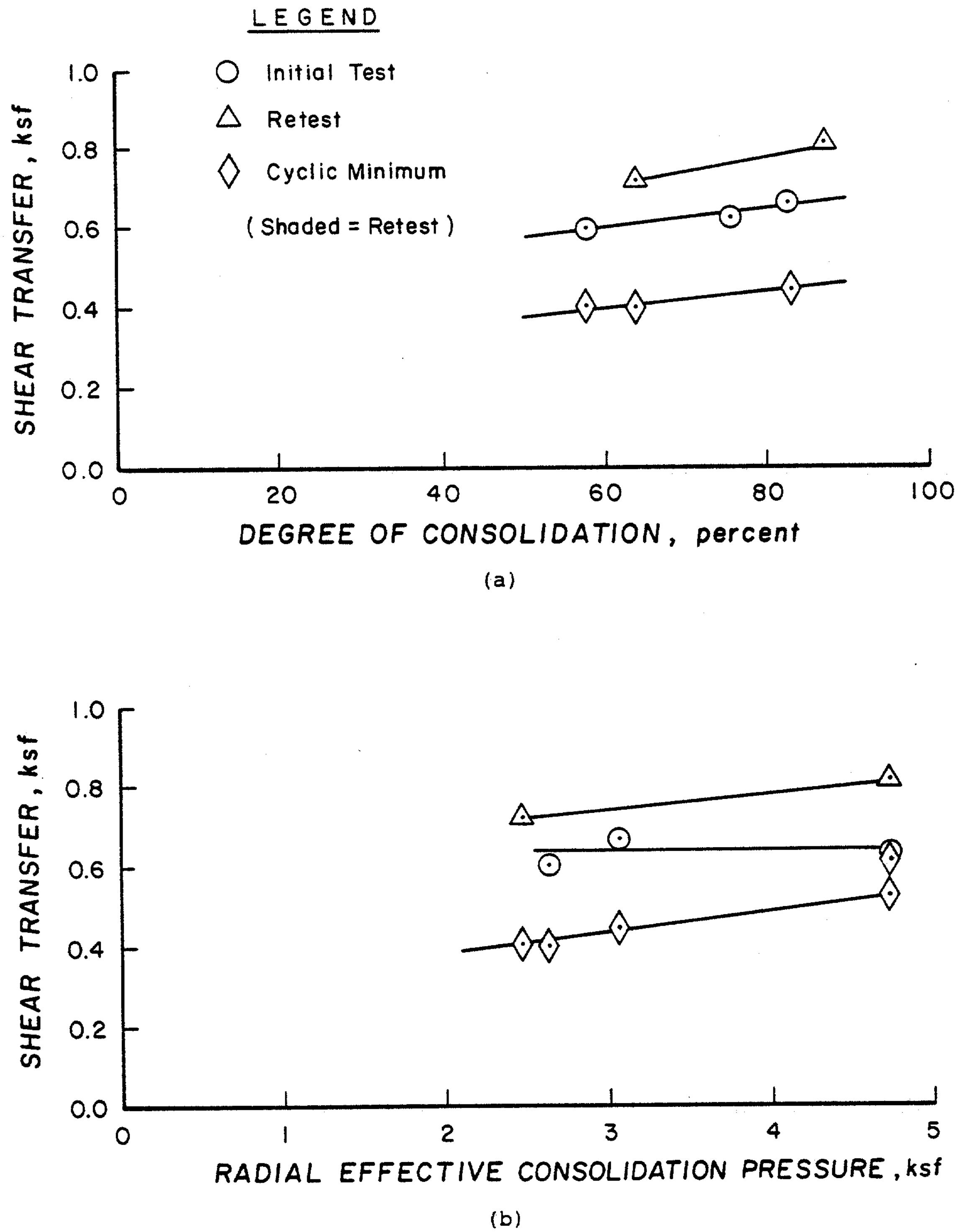
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



CYCLIC MINIMUM SHEAR TRANSFER AT 178 FT

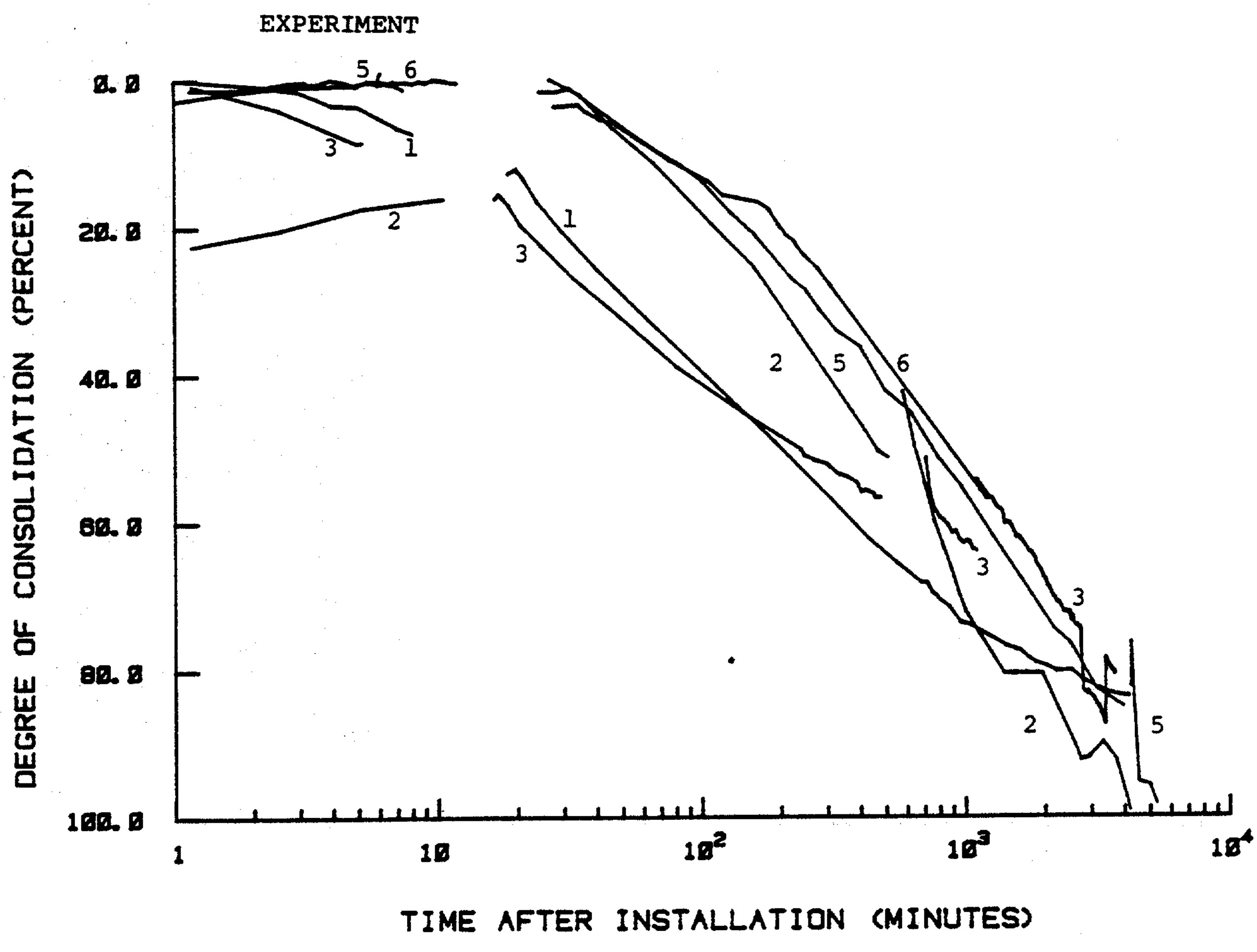
LIMITING SHEAR TRANSFER AFTER CYCLIC DEGRADATION

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



SUMMARY OF SHEAR TRANSFER AND PRESSURE DATA AT THE 178-FT DEPTH

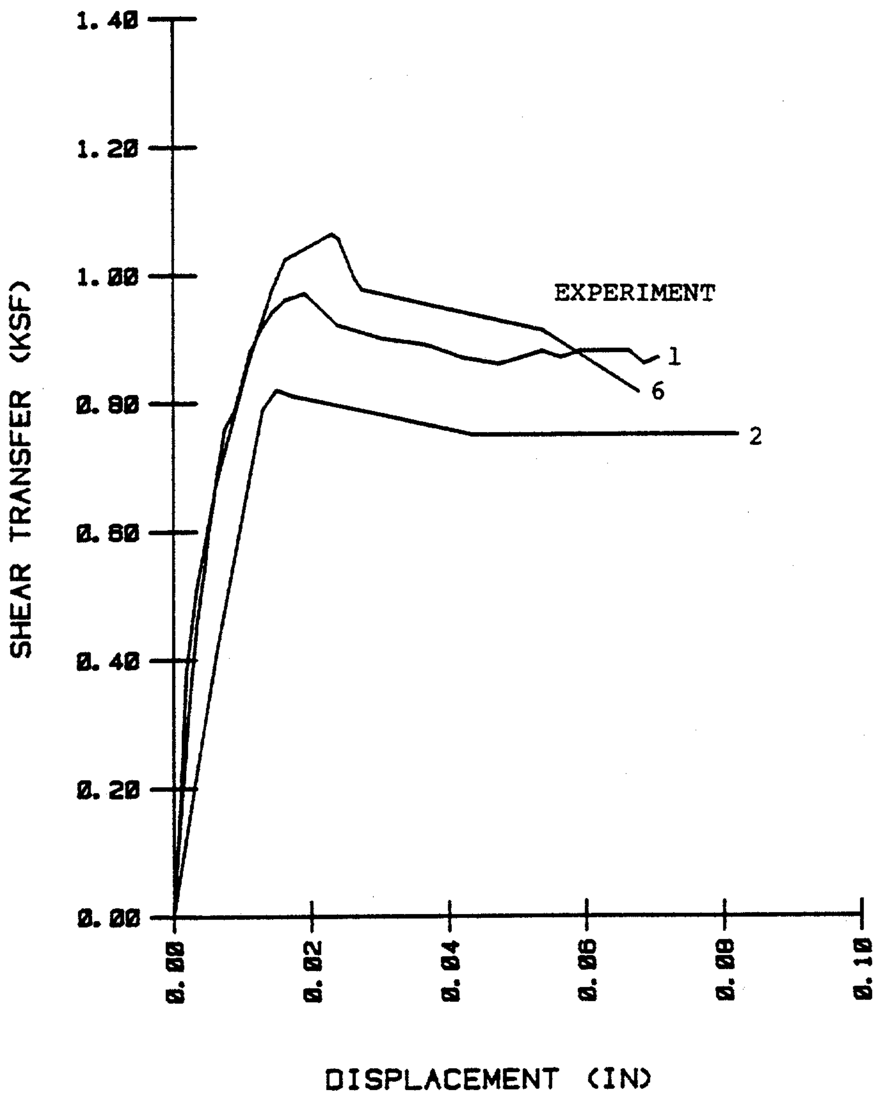
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



CONSOLIDATION AT 208 FEET

NORMALIZED PORE PRESSURE RESPONSE AT THE 208-FT DEPTH

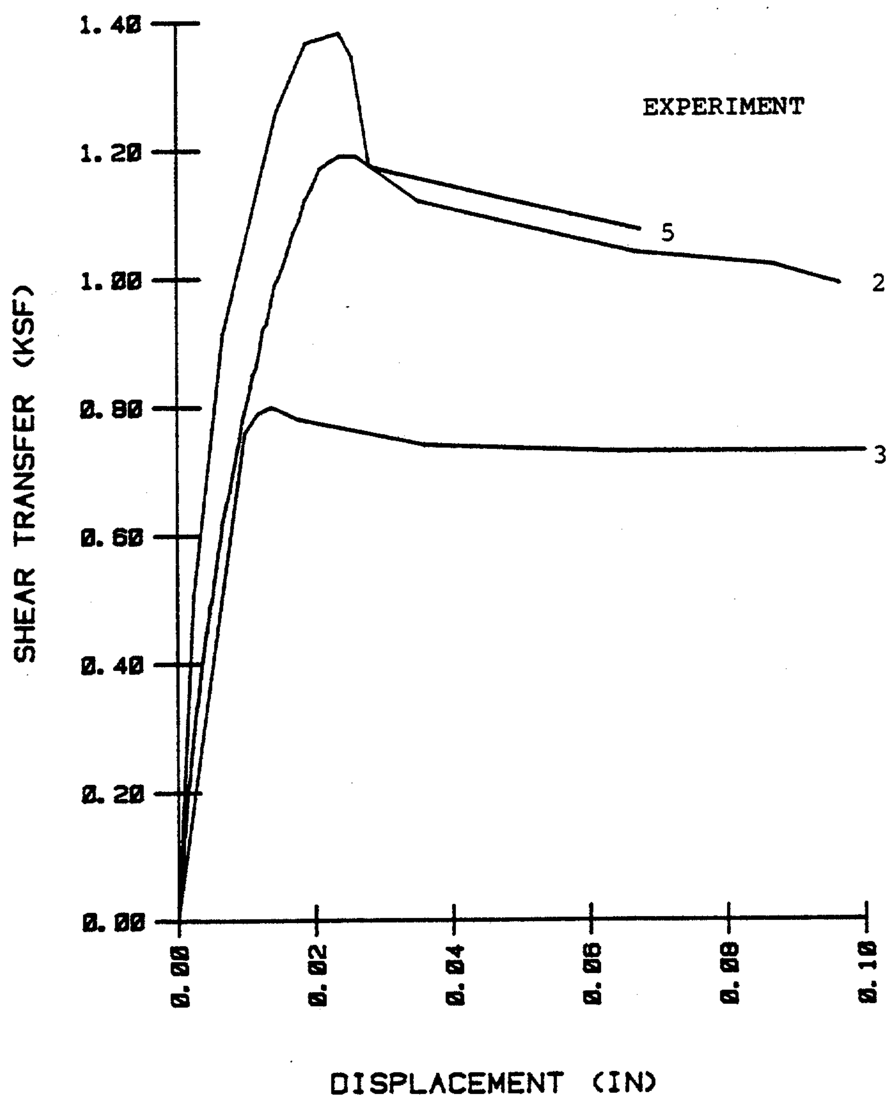
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



INITIAL TESTS AT 208 FEET

MAXIMUM SHEAR TRANSFER AFTER UNDISTURBED CONSOLIDATION

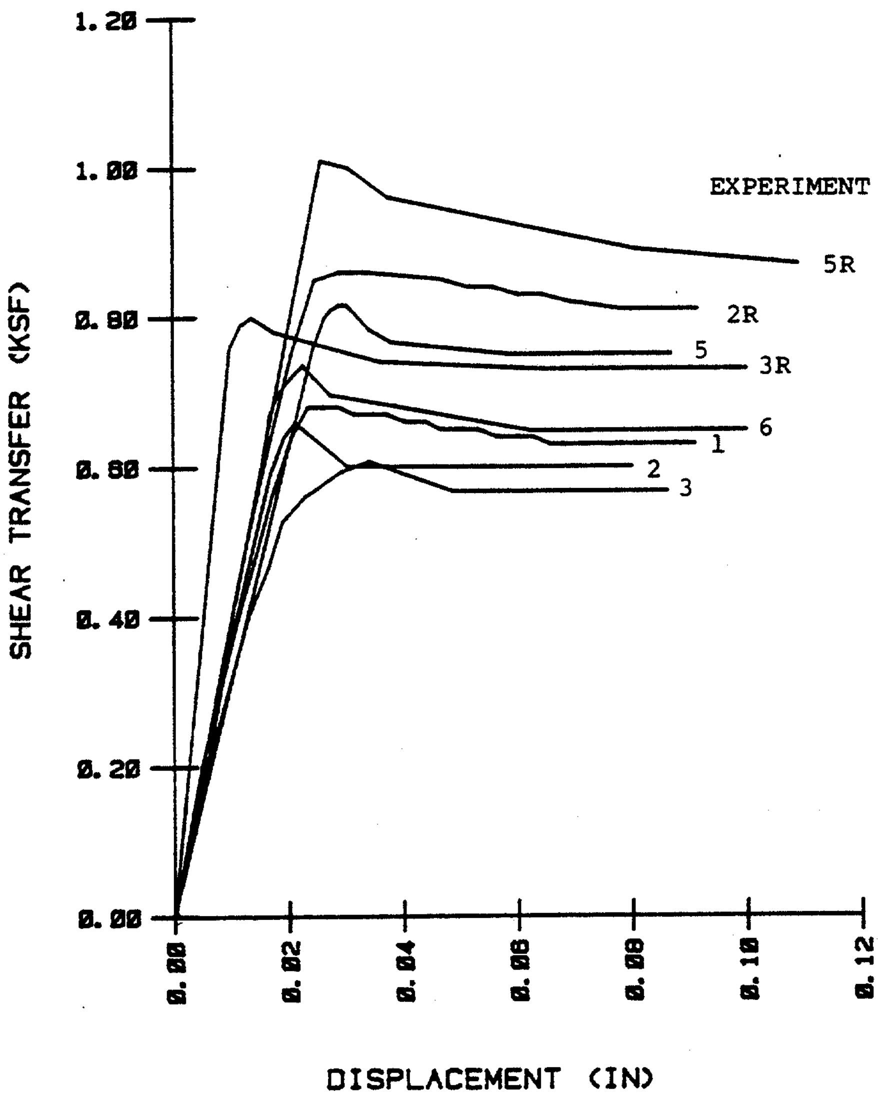
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



RETESTS AT 208 FEET

MAXIMUM SHEAR TRANSFER AFTER CYCLIC DEGRADATION AND RECONSOLIDATION

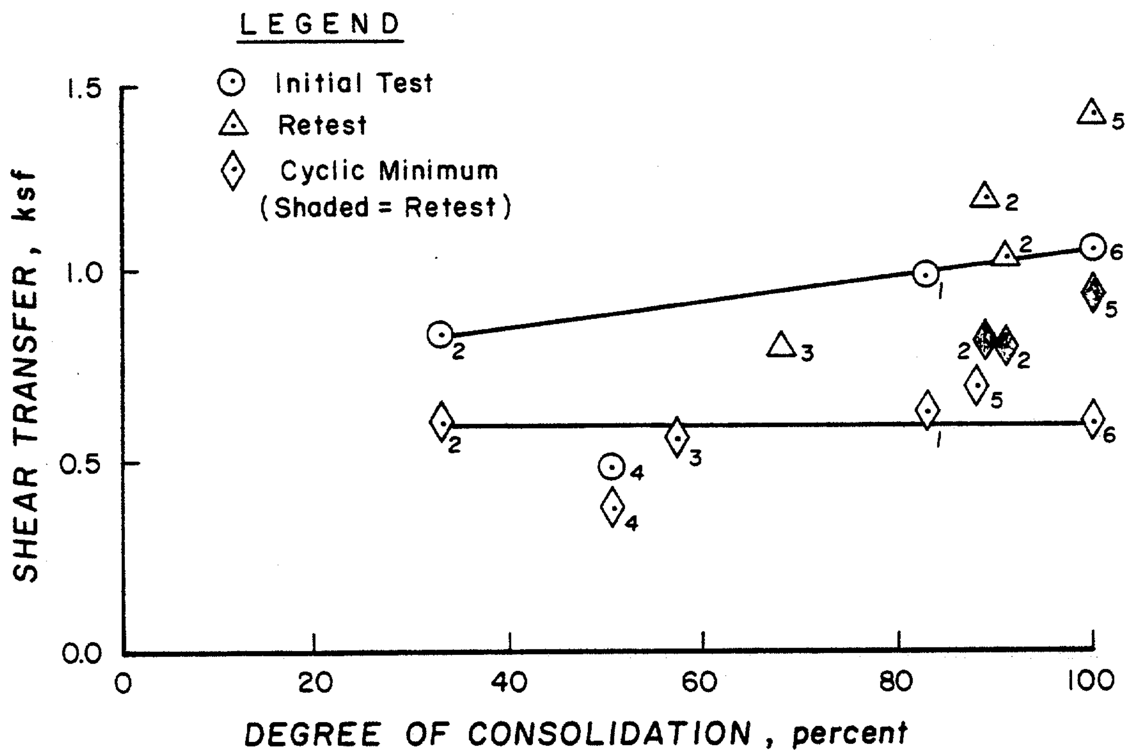
(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



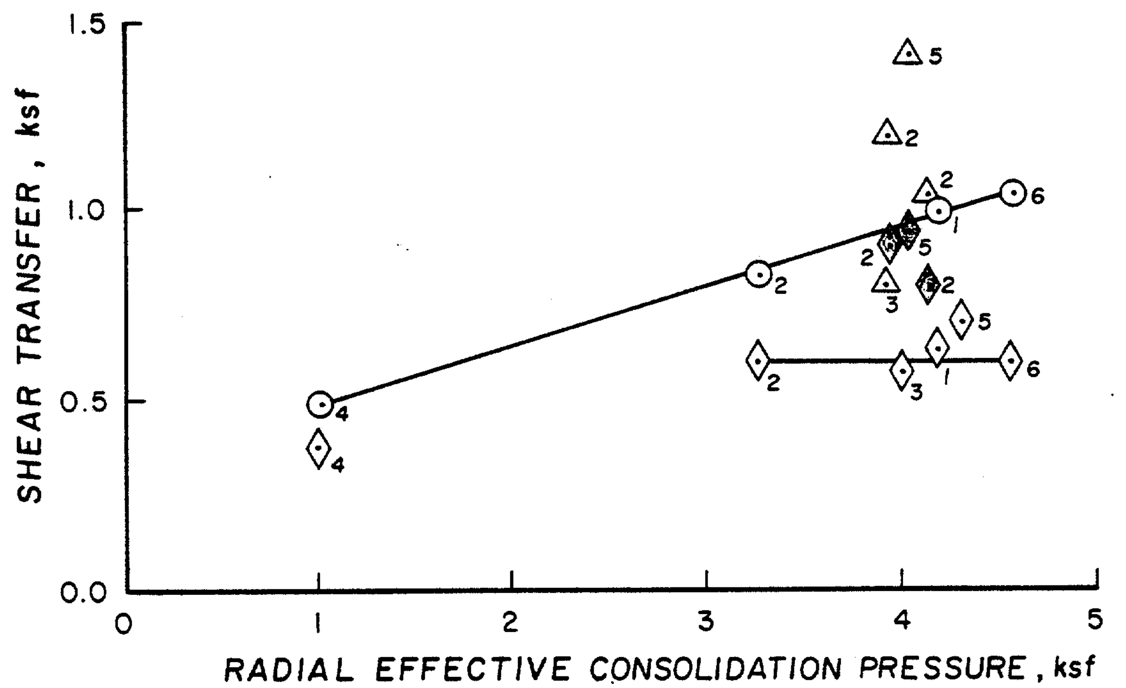
CYCLIC MINIMUM SHEAR TRANSFER AT 208 FT

LIMITING SHEAR TRANSFER AFTER CYCLIC DEGRADATION

(1 inch = 25.4 mm, 1 ft = 0.305 m, 1 kip = 4.45 kN, 1 ksf = 47.9 kPa)



(a)



(b)

SUMMARY OF SHEAR TRANSFER AND PRESSURE DATA AT THE 208-FT DEPTH

PLATE 6.19